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PROCEEDINGS

5th UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

TOKYO, JAPAN
APRIL 18-22, 1977
U.S. ENVIRONMENTAL PROTECTION AGENCY

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16. ABSTRACT

As part of joint interests in environmental matters between the United States and Japan, a Conference on Sewage Treatment Technology is held at intervals of about 18 months. This publication contains papers from the Japanese group and from the American side that were presented at the Fifth Conference. Subject matter covered includes application of P.L. 92-500 with a state program, regional wastewater plans, non-point source control, urban run-off, wastewater reuse, industrial wastewater pretreatment and joint treatment, waste treatability, nitrogen and phosphorus control, water reclamation technology, water pollution control (Japan), storm and combined sewer overflow, sludge treatment, automatic monitoring, and advanced waste treatment.

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PROCEEDINGS

FIFTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

APRIL 18-22, 1977
TOKYO, JAPAN

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FOREWORD

Environmental improvement is a worldwide need. Maintaining clean water supplies and managing municipal and industrial wastes is a vital element of a quality environment.

The participants in the United States-Japan cooperative project on sewage treatment technology have completed their fifth conference. These conferences, held at 18-month intervals, give the scientists and engineers of the cooperating agencies an opportunity to study and compare the latest practices and developments in the United States and Japan. These Proceedings of the Fifth Conference comprise a useful body of knowledge on sewage treatment, which will be available not only to Japan and the United States but also to any nation of the world which may wish to have it.

Douglas M. Costle
Administrator

Washington, D.C.

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UNITED STATES AND JAPAN
DELEGATES TO THE 5TH CONFERENCE

U.S. DELEGATES CONFERRING ON TREATMENT TECHNOLOGY
WITH MR. R. OKUDA, GOVERNOR OF NARA PREFECTURE

UNITED STATES TEAM INSPECTS
DEEP TUNNEL STORMWATER CONTROL PROJECT
OSAKA, JAPAN

UNITED STATES TEAM TOURS
ADVANCED WASTE TREATMENT PILOT PLANT
KYOTO, JAPAN

VIEWING A NEW SEWAGE TREATMENT PLANT
NARA, JAPAN

DR. M. KASHIWAYA AND DR. K. MURAKAMI
MINISTRY OF CONSTRUCTION, JAPAN

JOINT COMMUNIQUE

FIFTH US/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

TOKYO, JAPAN APRIL 28, 1977

1. The Fifth United States - Japan Conference on Sewage Treatment Technology was held in Tokyo, Japan from April 26 ~ 28, 1977.

2. The United States Delegation headed by Mr. F. M. Middleton, Senior Science Advisor, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, Ohio, was composed of six USEPA officials and two local government officials.

3. Mr. K. Inomaye, Director, Department of Sewerage and Sewage Purification, Ministry of Construction, was Head of the Japanese Delegation, other delegation members were three national government officials, two Japan Sewage Works Agency officials and three local government officials.

4. The Chairmanship for the Conference was shared jointly be Mr. F. M. Middleton and Dr. T. Kubo, Vice President, Japan Sewage Works Agency.

5. Prior to the Conference, the United States Delegation visited the Toba Sewage Treatment Plant, Kyoto; the Advanced Waste Treatment Pilot Plant, Otsu; the Takagi Tannery Wastewater Treatment Plant, Himeji; the Yamato River Purification Center, Nara; the Kawamata Sewage Treatment Plant and the Nakahama Sewage Treatment Plant, Osaka; the Morigasaki Sewage Treatment Plant, Tokyo and the South Sewage Treatment Plant, Yokohama. Each field visit involved the subject matter discussed in the Japanese side papers presented at the Conference so that the Conference may promote vigorous discussion more in detail.

6. During the Conference the United States Delegation presented a series of papers on the topics of Control of Non-point Source Pollution, Use of New Technology in the EPA Construction Grant Program, Georgia's Water Quality Control Program, Federal-State-Regional Participation in the Development of a Wastewater Management Plan, Regional Domestic Wastewater Management, Urban Run-off Pollution Control Technology, Wastewater Reuse, Industrial Wastewater Pretreatment and Joint Treatment, Criteria

and Assessment of Waste Treatability, Biological, Nitrogen and Phosphorus Control, and Water Reclamation Technology.

7. The Japanese side described Industrial Wastewater Control into Public Sewers, Recent Topics in Water Pollution Control in Japan, Storm and Combined Sewer Overflows, Regeneration of Granular Activated Carbon, Deep Aeration Tanks, Upgrading Existing Plant by Chemical Addition to Aeration Tanks, Rapid Sand Filtration Process for Tertiary Purpose. Papers on Sludge Disposal and Automatic Water Quality Monitoring Equipment were presented at the Conference as an interim report of Joint Research Works between the United States and Japan under the Joint Agreement of the US/Japan Conference on Sewage Treatment Technology.

8. Each presentation was followed by lively discussions from both sides.

9. Recent personnel exchanges include a short visit to Japan by Mr. J. T. Rhett and Mr. M. B. Cook, USEPA, Washington, D.C. to investigate institutional structures in the field of water pollution control in Japan. Mr. F. M. Middleton visited Japan to discuss progress of joint research works and the program for the Fifth Conference. Mr. K. Tanaka and Mr. S. Hiromoto, Japan Sewage Works Agency, are planning to spend several months in the United States in 1977 to study plant design and operation, and urban run-off pollution control technology respectively.

10. In addition to the Conference the Seminar was opened to about 200 members of the Japan Sewage Works Association on the subjects of the United States presentations during the Conference.

11. The Conference concluded that the technology exchange program including the Conference was fruitful to both sides in exchanging knowledge and experience and the Delegations agreed to seek to explore more effective cooperation in the field of research works and that the personnel exchange program should be continued.

12. It was proposed by the United States side that the Sixth Conference should be held in the United States, about October 1978.

FIFTH US/JAPAN CONFERENCE
ON
SEWAGE TREATMENT TECHNOLOGY
PAPER NO. 1

RECENT PROGRESS IN INDUSTRIAL WASTEWATER CONTROL DISCHARGED INTO PUBLIC SEWER

RECENT TOPICS IN WATER POLLUTION CONTROL IN JAPAN

APRIL 26-28, 1977
TOKYO, JAPAN

MINISTRY OF CONSTRUCTION
JAPANESE GOVERNMENT

JAPANESE PAPERS

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RECENT PROGRESS IN INDUSTRIAL WASTEWATER CONTROL DISCHARGED INTO PUBLIC SEWER

1. REASONS FOR REVISION OF THE SEWERAGE LAW

The public sewerage system is indispensable as a facility not only for the improvement of the living environment but also for the maintenance of the water quality of the public water bodies, and therefore the improvement of the public sewerage system is urged. However, such an improvement would not fully serve its purpose, if the effluent from households and factories did not flow into the public sewerage system and remained on the ground surface or continued to run along the conventional open sewers.

For this reason, the Sewerage Law provides that as the public sewerage system is opened for public use, all sewage in the relevant drainage area must be discharged into the public sewerage system with the exception of the effluent whose direct discharge into the public water bodies is approved by the general manager of the public sewerage system. The Law also provides that the water quality of the effluent discharged from the public sewerage system to rivers and other public water bodies must meet technical standards (Table 1) as specified under Article 8 of the Sewerage Law.

Therefore, it is the general principle that the following type of effluent discharged from factories in the drainage area is treated at their source to meet certain standards. The effluent subject to the treatment is one that could either disrupt the functions of sewerage facilities or damage the facilities or one that could make it difficult for the water quality of the effluent discharged from the final treatment facilities to meet technical standards as provided under Article 8 of the Law.

Table 1 Standards for Effluent Quality

Item Classification	Hydrogen-Ion Concentration (Hydrogen Exponent)	Biochemical Oxygen Demand (mg/ℓ in five days)	Amount of Suspended Solids (mg/ℓ)	No. of Bacteria Coli (No./cm^3)
Treatment of sewage by high-rate trickling filter process, modified activated sludge process and other processes with similar efficiency.	5.8~8.6	Less than 20	Less than 70	Less than 3000
Treatment of sewage by high-rate trickling filter process, modified activated sludge process and other processes with similar efficiency	5.8~8.6	Less than 60	Less than 120	Less than 3000
Treatment of sewage by sedimentation process	5.8~8.6	Less than 120	Less than 150	Less than 3000
Other	5.8~8.6	Less than 150	Less than 200	Less than 3000

Figures were calculated by the procedures provided for in the Ordinance of the Ministry of Construction and the Ordinance of the Ministry of Health and Welfare.

Table 1 (Continued)

Substances	Permissible Limits
Cadmium and Its Compounds (Cd)	0.1 mg/ℓ
Cyanide Compounds (CN)	1 mg/ℓ
Organic Phosphorus Compounds (Parathion, Methyl Parathion, Methyl Demeton and EPN only)	1 mg/ℓ
Lead and Its Compounds (Pb)	1 mg/ℓ
Chromium (VI) Compounds [Cr(VI)]	0.5 mg/ℓ
Arsenic and Its Compounds (As)	0.5 mg/ℓ
Total Mercury (Hg)	0.005 mg/ℓ
Alkyl Mercury Compounds	Not detectable[1]
PCB	0.003 mg/ℓ
N-hexane Extracts	5 mg/ℓ (mineral oil) 30 mg/ℓ (animal and vegetable fats)
Phenols	5 mg/ℓ
Copper (Cu)	3 mg/ℓ
Zinc (Zn)	5 mg/ℓ
Dissolved Iron (Fe)	10 mg/ℓ
Dissolved Manganese (Mn)	10 mg/ℓ
Chrome (Cr)	2 mg/ℓ
Fluorine (F)	15 mg/ℓ

In reference to controls on the inferior sewage discharged by factories or establishments that could disrupt the functions of sewerage facilities, damage them or aggravate the effluent quality of the final treatment facilities, the Sewerage Law before revision provided under Article 12 that the general manager of the public sewerage system under relevant regulations could order factories and other establishments to build pre-treatment facilities or take other necessary measures. Standards for the installation of pre-treatment facilities were to be instituted by city, town or village authoritie's regulations in accordance with the standards as prescribed by Cabinet Order (Table 2).

In this case, the standards provided by the regulations were the minimum requirements to maintain the structure and functions of the public sewerage system and conform the water quality of the effluent to the technical standards provided under Article 8 of the Law, and unduly obligations must not be imposed on the user of the public sewerage system.

However, it was pointed out that under the Sewerage Law before revision, guidance for the installation of sewage pre-treatment facilities tended to be delayed due to the following reasons.

a. The general manager of the public sewerage system was obliged to order the violator of the regulations to install pre-treatment facilities, but the penalty provisions were not applied unless he disobeyed the order given. The Law had no provisions for the so-called "direct penalty system" in which the violator of the regulations is punished immediately upon the revelation of his violation.

b. The Law provided no prior checking system as to the construction or renovation of facilities that could discharge inferior sewage.

Table 2 Standards of Regulation Concerning Installation of Pre-treatment Facilities

Substances	Permissible Limits
Temperature	45°C
pH	5~9
BOD	600 mg/ℓ
SS	600 mg/ℓ
N-hexane Extracts	
Mineral Oils	5 mg/ℓ
Animal and Vegetable Fats	30 mg/ℓ
Consumption of Iodine	220 mg/ℓ
Phenols	5 mg/ℓ
Cyanide Compounds (CN)	1 mg/ℓ
Alkyl Mercury Compounds	ND
Organic Phosphorus Compounds	1 mg/ℓ
Cadmium and Its Compounds (Cd)	0.1 mg/ℓ
Lead and Its Compounds (Pb)	1 mg/ℓ
Chromium (VI) Compounds [Cr (VI)]	0.5 mg/ℓ
Arsenic and Its Compounds (As)	0.5 mg/ℓ
Total Mercury (Hg)	0.005 mg/ℓ
Chrome (Cr)	2 mg/ℓ
Copper (Cu)	3 mg/ℓ
Zinc (Zn)	5 mg/ℓ
Dissolved Iron (Fe)	10 mg/ℓ
Dissolved Manganese (Mn)	10 mg/ℓ
Fluorine (F)	15 mg/ℓ
PCB	0.003 mg/ℓ

With respect to sanitary sewage discharged from facilities used for manufacturing and gas supplying business, the standards can be tightened up to those at the table below, if it is recognized that the total amount of sanitary sewage from them is equivalent to more than one-fourth of sanitary sewage to be treated at the treatment facility, that it will not be sufficiently diluted by other sanitary sewage before reaching the treatment facility or that there is any other justifiable reason.

Substances	Permissible Limits
Temperature	40°C
pH	5.7~8.7
BOD	300 mg/ℓ
SS	300 mg/ℓ

As a result, the Law was revised to improve on the above-mentioned points, strengthen controls on the effluent from factories or establishments and thus contribute to the maintenance of the water quality of the public water bodies.

2. SUMMARY OF REVISION IN THE SEWERAGE LAW

2.1 INTRODUCTION OF DIRECT PENALTY SYSTEM

Under the revised Sewerage Law, provisions less effective than the Water Pollution Control Law were improved, and a new provision was introduced to prevent the discharge of the sewage whose water quality does not meet specified legal standards. Violations, including accidental violations, are now subject to immediate punishment on the revelation of their violations without taking supervisory action for them. Thus the revised Law has adopted the so-called "direct penalty system."

This new provision, as in the Water Pollution Control Law, is applied to factories or establishments which operate specified facilities (as designated under the Water Pollution Control Law as facilities that could discharge inferior waste water – Table 3). But its application excludes kitchen facilities, washing facilities and bathing facilities (not using water from hot springs) of hotel business and the final sewerage treatment facilities.

Table 3 Specified Facilities

(1) MINING AND COAL WASHING
(a) ore separation facilities, (b) coal dressing facilities, (c) neutralization and sedimentation facilities of mine water, (d) solids separation facilities from water used for digging.

(1)-2 LIVESTOCK BREEDING
(a) pig shed facilities (excluding the facilities installed in the shed with the total area of less than 50 m^2)
(b) cattle shed facilities (excluding the facilities installed in the shed with the total area of less than 200 m^2)
(c) horse shed facilities (excluding the facilities installed in the shed with the total area of less than 500 m^2)

(2) MEAT PACKING AND POULTRY PROCESSING
(a) initial preparation facilities, (b) washing facilities, (c) cooking facilities.

(3) SEA FOODS MANUFACTURING
(a) initial preparation facilities, (b) washing facilities, (c) dehydration facilities, (d) screening facilities, (e) cooking facilities.

(4) CANNED AND FROZEN VEGETABLES AND FRUITS MANUFACTURING
(a) initial preparation facilities, (b) cleaning facilities, (c) pressing facilities, (d) cooking facilities.

(5) MISO, SOY-SOURCE, EDIBLE AMINO ACID, GLUTAMIC ACID, VEGETABLE SOURCES AND VINEGAR MANUFACTURING
(a) initial preparation facilities, (b) cleaning facilities, (c) boiling facilities, (d) concentration facilities, (e) finishing facilities, (f) straining facilities.

(6) WHEAT FLOUR MANUFACTURING
(a) washing facilities.

(7) SUGAR MANUFACTURING
(a) initial preparation facilities, (b) washing facilities, (c) filtration facilities, (d) separation facilities, (e) refining facilities.

(8) BAKERY AND CONFECTIONARY
(a) bean-jam processing facilities.

(9) RICE CAKE AND MALT MANUFACTURING
(a) washing facilities.

(10) SOFT DRINK MANUFACTURING AND BREWERY
(a) initial preparation facilities, (b) cleaning facilities, (c) extraction facilities, (d) straining facilities, (e) boiling facilities, (f) stilling facilities.

(11) FEED STAFF AND ORGANIC FERTILIZER MANUFACTURING
(a) initial preparation facilities, (b) washing facilities, (c) pressing facilities, (d) vacuum concentration facilities, (e) water bushing deodorization facilities.

(12) OIL AND FAT MANUFACTURING
(a) initial preparation facilities, (b) washing facilities, (c) pressing facilities, (d) separation facilities.

(13) YEAST MANUFACTURING
(a) initial preparation facilities, (b) washing facilities, (c) separation facilities.

(14) STARCH MANUFACTURING
(a) soaking facilities, (b) washing facilities, (c) separation facilities, (d) waste pits.

(15) DEXTROSE MANUFACTURING
(a) initial preparation facilities, (b) filtration facilities, (c) refining facilities.

(16) NOODLES MANUFACTURING
(a) boiling facilities.

(17) BEAN FOODS MANUFACTURING
(a) boiling facilities.

(18) INSTANT COFFEE MANUFACTURING
(a) extraction facilities.

(19) TEXTILE INDUSTRY
(a) scouring facilities, (b) by-product processing facilities, (c) soaking facilities, (d) finishing facilities, (e) silket machine, (f) bleaching facilities, (g) dyeing facilities, (h) chemical treatment facilities.

(20) WOOL SCOURING AND WASHING
(a) wool scouring and washing facilities, (b) carbonizing facilities.

(21) SYNTHETIC TEXTILE MANUFACTURING
(a) spinning facilities, (b) chemical treatment facilities, (c) recovery facilities.

(22) CHEMICAL FINISHING OF WOODS
(a) wet barker, (b) chemical soaking facilities.

(23) PULP AND PAPER MANUFACTURING
(a) soaking, (b) wet barker, (c) chiper, (d) digester, (e) accumulater for digester waster, (f) chip refiner and pulp refiner, (g) bleaching facilities, (h) paper mill, (i) cellophane paper mill, (j) wet fiber plate facilities, (k) waste gas washing facilities.

(24) FERTILIZER MANUFACTURING

(a) filtration facilities, (b) separation facilities, (c) water jet breaking facilities, (d) waste gas washing facilities, (e) wet dust collector.

(25) SODIUM HYDROXIDE AND POTASSIUM HYDROXIDE MANUFACTURING (MERCURY ELECTROLYSIS)

(a) electrolyte refining facilities, (b) electrolyzing facilities.

(26) INORGANIC PIGMENT MANUFACTURING

(a) washing facilities, (b) filtration facilities, (c) centrifuger (cadmium and its compounds), (d) water flushing separate (erdigris), (e) waste gas washing facilities.

(27) INORGANIC CHEMICALS MANUFACTURING EXCLUDING ITEMS OF 25 AND 26

(a) filtration facilities, (b) centrifuger, (c) sulfur dioxide gas cooling and washing facilities (sulfuric acid), (d) washing facilities (activated carbon and carbonated disulfur), (e) hydrochloric acid regenerating facilities (silicate anhydrous), (f) reactor (cyanides), (g) absorber and sedimentation facilities (iodines), (h) sedimentation facilities (saline magnesia), (i) water flushing facilities (bariumates), (j) waste gas washing facilities, (k) wet dust collector.

(28) ETHYLENE DERIVATES MANUFACTURING (CARBIDE PROCESS)

(a) wet ethylene generation facilities, (b) washing facilities and still (acetate ester), (c) methyl alcohol still (polyvinyl alcohol), (d) still (acrylic acid ester), (e) vinyl chloric monomer washing facilities, (f) chloroprene monomer washing facilities.

(29) COAL TAR PRODUCTS MANUFACTURING

(a) sulwuric acid washing facilities of benzene relates, (b) waste pits, (c) tar sodium fulfonate reactor.

(30) FERMENTATION INDUSTRY EXCLUDING ITEMS OF 5, 10 AND 13

(a) initial preparation facilities, (b) still, (c) centrifugal decanter, (d) filtration facilities.

(31) METHANE DERIVATES MANUFACTURING

(a) still (methyl alcohol and 4-chloromethane), (b) refining facilities (folmaldehyde), (c) washing and filtration facilities (fione gas).

(32) SYNTHETIC PLASTIC MANUFACTURING

(a) filtration facilities, (b) water washing facilities (pigments or dye lake), (c) centrifugal decanter, (d) waste gas washing facilities.

(33) SYNTHETIC PLASTIC MANUFACTURING

(a) condensation reactor, (b) water washing facilities, (c) centrifugal decanter, (d) settling facilities, (e) cooling gas washer and still (florides plastics), (f) diluent still (polypropylene), (g) diluent still (polyethylene), (h) acid and alkali treatment facilities (polybutane), (i) waste gas washing facilities, (k) wet dust collector.

(34) SYNTHETIC RUBBER MANUFACTURING

(a) filtration facilities, (b) dehydration facilities, (c) washing facilities, (d) latex concentration facilities, (e) sedimentation facilities (styrene-butadiene, nitrile-butadiene and polybutadiene-gum).

(35) ORGANIC GUM CHEMICALS MANUFACTURING

(a) destillation facilities, (b) waste gas washing facilities, (c) wet dust collector.

(36) SYNTHETIC DETERGENT MANUFACTURING

(a) acid washing and separating facilities, (b) waste gas washing facilities, (c) wet dust collector.

(37) PETROCHEMICAL INDUSTRIES (CARBOHYDRATE AND ITS DERIVATES) EXCLUDING ITEMS FROM 31 TO 36 AND 51

(a) washing facilities, (b) separation facilities, (c) filtration facilities, (d) distillation and rapid cooling facilities (acrylonitrile), (e) distillation facilities (acetoaldehyde, eterphatalic acid, thylene diamine), (f) acid and alkali treatment facilities (alkyl benzene), (g) distillation facilities and sulphuric acid concentration facilities (iso-propyl alcohol), (iso-propyl alcohol), (h) distillation and condensation reactor (alcohol), (i) gas cooling and washing facilities (phalic acid anhydride), (j) acid and alkali treatment facilities (cyclohexane), (k) methylalcohol distillation facilities and acid, alkal treatment facilities, (l) steam condensor (ethylketone), (m) methylalcohol recovery facilities and reactor (methy-m-acrytate monomer), (p) waste gas washing facilities.

(38) SOAP MANUFACTURING

(a) initial preparation facilities, (b) salting out facilities.

(39) HYDROGENATED OIL MANUFACTURING

(a) alkali conditioning facilities, (b) deodorization facilities.

(40) FATTY ACIDS MANUFACTURING

(a) distillation facilities.

(41) PERFUMERY MANUFACTURING

(a) washing facilities, (b) extraction facilities.

(42) GELATINE AND GLUE MANUFACTURING

(a) initial preparation facilities, (b) lime soaking facilities, (c) washing facilities.

(43) PHOTO SENSITIVE GOODS MANUFACTURING

(a) washing facilities.

(44) NATURAL RESIN MANUFACTURING

(a) initial preparation facilities, (b) dehydration facilities.

(45) WOODS CHEMICAL MANUFACTURING

(a) furfural distillation facilities.

(46) ORGANIC CHEMICALS MANUFACTURING EXCLUDING ITEMS FROM 28 TO 45

(a) water washing facilities, (b) filtration facilities, (c) concentrator (hyrazide), (d) waste gas washing facilities.

(47) PHARMACEUTICAL MANUFACTURING
(a) initial preparation facilities, (b) filtration facilities, (c) separation facilities, (d) mixing facilities, (e) gas washing facilities.

(48) GUNPOWDER MANUFACTURING
(a) washing facilities.

(49) PESTICIDES MANUFACTURING
(a) mixing facilities.

(50) PEAGENT MANUFACTURING
(a) processing facilities.

(51) OIL REFINING INDUSTRY
(a) desalting facilities, (b) crude petroleum distillation facilities, (c) desulfurization facilities, (d) washing facilities (volatile oil, kerosene, or gasoline), (c) lubricant washing facilities.

(52) LEATHER MANUFACTURING
(a) washing facilities, (b) line soaking facilities, (c) tannin soaking facilities, (d) chrome bathing facilities, (e) dyeing facilities.

(53) CLASS MANUFACTURING
(a) grinding and washing facilities, (b) gas washing facilities.

(54) CEMENT MANUFACTURING
(a) centrifuger, (b) shaper, (c) wet conditioning facilities.

(55) READY MIXED CONCRETE MANUFACTURING
(a) batcher plant.

(56) ORGANIC SAND BOARD MANUFACTURING
(a) mixing facilities.

(57) SYNTHETIC BLACK LEAD ELECTRODE MANUFACTURING
(a) shaping facilities.

(58) RAW POTTERY MATERIALS MANUFACTURING
(a) water jet crusher, (b) separation facilities, (c) acid treatment facilities, (d) dehydration facilities.

(59) MACADAM QUARRING
(a) water jet crusher, (b) separation facilities.

(60) SAND COLLECTION

(61) IRON INDUSTRY
(a) tar and gas separation facilities, (b) gas coiling and washing facilities, (c) rolling facilities, (d) hardening facilities, (e) wet dust collector.

(62) NONFERREOUS METALS MANUFACTURING
(a) reduction basins, (b) electrolysis facilities, (c) hardening facilities, (d) mercury refinery facilities, (e) waste gas washing facilities, (f) wet dust collector.

(63) METRIC GOODS MANUFACTURING AND MACHINERY INDUSTRY
(a) hardening facilities, (b) surface treatment facilities electrolysis), (c) cadmium electrode and lead electrode processing facilities, (d) mercury refining facilities, (e) waste gas washing facilities.

(64) TOWN GAS AND COKE MANUFACTURING
(a) coal tar and gas-liquid separation facilities, (b) cooling and washing facilities including desulfurization facilities.

(64)-2 WATER CLEANING FACILITIES (excluding those whose cleaning capability is less than 10,000 m^3 a day) OF WATER SUPPLY FACILITIES (defined by the provision of paragraph 7 of Article 3 of the Water Supply Law), INDUSTRIAL WATER SUPPLY FACILITIES (defined by the provision of paragraph 6 of Article 2 of the Industrial Water Supply Business Law) or PRIVATE INDUSTRIAL WATER SUPPLY FACILITIES (defined by the provision of paragraph 1 of Article 21 of the same Law)
(a) depositing facilities, (b) filter facilities.

(65) (a) acid and alkali treatment facilities of metal surface.

(66) (a) electro-plating facilities.

(66)-2 LODGING SERVICE (defined by the provision of paragraph 1 of Article 2 of the Lodging Service Law, excluding boarding house service)
(a) cooking facilities, (b) bath facilities, (c) washing facilities.

(67) LAUNDRY
(a) washing facilities.

(68) PHOTO DEVELOPING
(a) automatically washing facilities of film.

(69) (a) slaughter house.

(69)-2 CENTRAL WHOLESALE MARKET FACILITIES (concerning marine production defined by the provision of paragraph 3 of Article of the Wholesale Market Law)
(a) wholesale market, (b) intermediate wholesale market.

(70) (a) waste oil treatment facilities.

(71) (a) automatically washing facilities of car.

(71)-2 RESEARCH, DETERMINATION, MEASUREMENT OR PROFESSIONAL EDUCATION FOR SCIENCE AND TECHNOLOGY (excluding human science)
(a) washing facilities, (b) hardening facilities.

(72) (a) night soil treatment plan (more than 501 population served).

(73) (a) sewage treatment plant.

(74) (a) waste water treatment plant.

The water quality standards set to prevent the discharge of sewage into the public sewerage system are designed to conform the water quality of the effluent to the technical standards provided under Article 8 of the Law. They are divided into two categories. One is to designate substances to be controlled for a nationwide uniform application under Cabinet Order. The other group of standards is set by regulations instituted by the general manager of the public sewerage system taking into account the types and distribution of factories in a given area, the capacities of the final treatment facilities in the area and other factors. As to the items which could damage sewerage facilities, such as the temperatures, hydrogen ion concentrations, the content of normal hexan extracts and iodic consumption, the general manager of the public sewerage system, as in the past, will decide obligations for installing pre-treatment facilities by regulations, and the direct penalty provisions are not introduced from the standpoint of functional damage.

The standards established by Cabinet Order for a nationwide uniform application (Table 4) provide for copper, zinc and other substances which are usually difficult to treat at the final sewage treatment facilities. These are picked from the list of substances provided for under the Water Pollution Control Law as substances dangerous to human health, such as cadmium and cyanide (health items) and those that could adversely affect the living environment, such as copper and zinc (living environment items). The control values are identical to the standards provided for under the Water Pollution Control Law and regulations instituted thereunder.

Table 4 Uniform Standards Set by Cabinet Order

Substances	Permissible Limits
Cadmium and Its Compounds (Cd)	0.1 mg/ℓ
Cyanide Compounds (CN)	1 mg/ℓ
Organic Phosphorus Compounds (Parathion, Methyl Parathion, Methyl Demeton and EPN only)	1 mg/ℓ
Lead and Its Compounds (Pb)	1 mg/ℓ
Chromium (VI) Compounds [Cr (VI)]	0.5 mg/ℓ
Arsenic and Its Compounds (As)	0.5 mg/ℓ
Total Mercury (Hg)	0.005 mg/ℓ
Alkyl Mercury Compounds	Not detectable
PCB	0.003 mg/ℓ
Phenols	5 mg/ℓ
Copper (Cu)	3 mg/ℓ
Zinc (Zn)	5 mg/ℓ
Dissolved Iron (Fe)	10 mg/ℓ
Dissolved Manganese (Mn)	10 mg/ℓ
Chrome (Cr)	2 mg/ℓ
Fluorine (F)	15 mg/ℓ

As to pH, BOD and other substances that can be disposable at the final treatment facilities, the general manager of the public sewerage system, as in the past, can decide, through the enactment of regulations, the water quality standards within the limits provided for under Cabinet Order taking into consideration the treatment capacity of each facility and other factors.

Under the Water Pollution Control Law, exceptions are provided for living environment items, such as that factories or establishments whose average daily discharge of the effluent is less than 50 m^3 are exempted from direct penalty provisions relevant to the sewage standards. With a view to coordinating relevant provisions with the Water Pollution Control Law, the sewage exempted from application of the Water Pollution Control Law are placed outside controls by the Sewerage Law. But the controls are applied to those factories or establishments whose average daily discharge is less than 50 m^3 when they are also controlled by regulations enforced under the Water Pollution Control Law.

Table 5 Limit of Standards Allowed to be Set by the Regulation

Substances	Permissible Limits
pH	5~9
BOD	600 mg/ℓ
SS	600 mg/ℓ
N-hexane Extracts	
Mineral Oil	5 mg/ℓ
Animal and Vegetable Fats	30 mg/ℓ

With respect to sanitary sewage discharged from facilities used for manufacturing and gas supplying business, the standards can be tightened up to those at the table below, if it is recognized that the total amount of sanitary sewage from them is equivalent to more than one-fourth of sanitary sewage to be treated at the treatment facility, that it will not be sufficiently diluted by other sanitary sewage before reaching the treatment facility or that there is any other justifiable reason.

Substances	Permissible Limits
pH	5.7~8.7
BOD	300 mg/ℓ
SS	300 mg/ℓ

Those who have violated this direct penalty provision are liable to penal servitude for less than six months or a fine of less than ¥200,000 (in accidental cases imprisonment for less than three months or a fine of less than ¥100,000).

The aforementioned controls are applied only to the specified factories. But when the sewage from non-specified factories is inferior, the general manager of the public sewerage system, as in the past, can take necessary measures to them by regulations, including an order for the installation of pre-treatment facilities.

2.2 INTRODUCTION OF A PRIOR CHECKING SYSTEM

Another major point of revision is the establishement of a prior checking system in which the general manager of the public sewerage system can examine in advance plans for the construction or renovation of specified facilities which could discharge inferior sewage. The details are followng.

2.2.1 NOTIFICATION OF CONSTRUCTION OF SPECIFIED FACILITIES

Persons who use the public sewerage system to discharge sewage from their factories or establishments must submit notifications to the general manager of the public sewerage system, when they build specified facilities or take other measures on them.

This provision enables the general manager of public sewerage system to fully examine in advance the construction and renovation of specified facilities by obliging the operators to submit their plans in advance to the general manager of the public sewerage system and by ordering modifications. Matters subject to notification are as follows:

a. Name or title and address, and name of the representative if factory or establishment is a corporate body

b. Name of factory or establishment and its location
c. Sort of specified facilities
d. Structure of specified facilities
e. Method of use of specified facilities
f. Method of treating sewage discharged from specified facilities
g. Volume, water quality of sewage discharged to public sewerage system and other matters specified by the Ordinance of the Ministry of Construction

2.2.2 ORDERS FOR MODIFICATION OF PLANS

When a notification for the construction of specified facilities is submitted the general manager of the public sewerage system can order, only within 60 days from the date of notification, the modification of the structure of the relevant specified facilities and of the sewage treatment method, if the water quality of sewage to be discharged to the public sewerage system is not deemed to conform to the discharge standards. During this period, construction work and other procedures concerning the specified facilities are to be prohibited to be done. In ordering the modification of the plan, the general manager can order the abrogation of the plan, if he considers it impossible to prevent the inflow to the sewerage system of the sewage unable to meet the control standards through the modification of the plan.

However, even within 60 days from the date of notification, the general manager of the public sewerage system can approve of the construction and other measures for the specified facilities if he considers the content of the notified plan appropriate.

2.3 INTRODUCTION OF IMPROVEMENT ORDER SYSTEM

Along with the prior checking system allowing orders to be given to modify the notified plan for the construction of specified facilities, the general manager of the public sewerage system can order the operator of specified facilities to improve their structure, their method of sewage treatment, etc. in order to check in advance the inflow of inferior sewage to the sewerage system, when he considers that the relevant facilities could discharge sewage that fails to meet the control standards. The improvement order includes the improvement of the structure of specified facilities and the suspension of the use of specified facilities and of the discharge of sewage for a certain period. The revision of the Sewerage Law as mentioned above will go into force as of May 1, 1977. But the sewage discharged from the existing specified factories will be exempted from the direct penalty and improvement order provisions for six months from the date of enforcement of the revised law (the period of exemption is one year for facilities designated under Cabinet Order).

3. PRESENT STATE OF CONTROLS ON FACTORY EFFLUENT

3.1 PRESENT STATE

3.1.1 NUMBER OF SPECIFIED FACTORIES

Through the introduction of a prior checking system under the revised Sewerage Law, persons who plan to construct specified facilities will be obliged to notify the general manager of the public sewerage system of thier content and other matters. But even the existing law requires a notification to the general manager of the public sewerage system from those who will establish specified facilities or discharge sewage in quantities as specified (more than 50 m³ per day) or of water quality as specified (Table 2) under Cabinet Order.

The numbers of specified factories (factories and other establishments equipped with specified facilities) within specific sewered areas as of March 31, 1976 are as listed on Table 6.

And the numbers of those who submited prior notification except those who established specified facilities are as listed on Table 7.

Table 6 Numbers of Specified Factories

Classification	Municipalities	Number of Specified Factories	Number of Required Pretreatment Facilities (A)	Number of Installed Pretreatment Facilities (B)	(B)/(A)%
Plants Generated Related Pollution Materials Related Health Hazard	Major Cities*) (10)	4.813	4,469	3,672	82.2
	Ordinary Cities	1,668	1,143	921	80.6
	Sub-total	6,481	5,612	4,593	81.8
Plants Generated Pollution Material Related to Living Environment, Discharging more than 50 m³/day of Waste Water	Major Cities (10)	1,298	717	419	58.4
	Ordinary Cities	1,497	877	608	69.3
	Sub-total	2,795	1,594	1,027	64.4
Plants Generated Pollution Material Related to Living Environment, Discharging less than 50 m³/day of Waste Water	Major Cities (10)	22,480	926	450	48.6
	Ordinary Cities	18,594	3,448	1,535	44.5
	Sub-total	41,074	4,374	1,985	45.4
Miscellaneous	Total for Major Cities	28,591	6,112	4,541	74.3
	Total for Ordinary Cities	21,759	5,468	3,064	56.0
	Total	50,350	11,580	7,605	65.7

*) Major Cities: Tokyo, Osaka, Sapporo, Yokohama, Kawasaki, Nagoya, Kyoto, Kobe, Kitakyushu, Fukuoka.

Table 7 Non-Specified Factories

Classification	Municipalities	Number of Required Pretreatment Facilities (A)	Number of Installed Pretreatment Facilities (B)	(B)/(A)%
Plants Generated Related Pollution Materials Related Health Hazard	Major Cities*) (10)	57	35	61.4
	Ordinary Cities	73	46	63.0
	Sub-total	130	81	62.3
Plants Generated Pollution Material Related to Living Environment, Discharging less than 50 m³/day of Waste Water	Major Cities (10)	157	116	73.9
	Ordinary Cities	195	115	59.0
	Sub-total	352	231	65.6
Plants Generated Pollution Material Related to Living Environment, Discharging less than 50 m³/day of Waste-Water	Major Cities (10)	381	289	75.9
	Ordinary Cities	1,307	459	35.1
	Sub-total	1,688	748	44.3
Miscellaneous	Total for Major Cities	595	440	73.9
	Total for Ordinary Cities	1,575	620	39.4
	Total	2,170	1,060	48.8

*) Major Cities: Tokyo, Osaka, Sapporo, Yokohama, Kawasaki, Nagoya, Kyoto, Kobe, Kitakyushu, Fukuoka.

On Table 8 are the numbers of specified factories that come under the Water Pollution Control Law. Under the Law the uniform sewer standards on BOD and other living environment-related items are applied to those specified factories whose average daily discharge exceeds 50 m³ and health-related items are applied to all specified factories. The number of specified factories subject to the uniform sewer standards is 31,891 or 14% of the total. The corresponding number and percentage for those factories within specific sewer areas are 9,276 and 18%. The ratios of those discharging more than 50 m³ daily are about the same. And, water used by manufacturers during 1974 totaled 18.1 billion m³, and the total excluding the amount used for raw material was 16.9 billion m³. On the other hand, the amount of factory effluent flown into the final treatment facilities was 0.8 billion m³ or 18% of the total.

3.1.2 TYPES OF SPECIFIED FACTORIES

Table 10 shows the types of specified factories. Many of the specified factories are small in scale.

Table 8 Specified Facilities Designated Under the Water Pollution Control Law (in Fiscal 1974)

No. of the Table 1 in Cabinet Order	Kind of Business or Name of Facilities	Number of Specified Factories				Number of Specified Factories to be Controlled by the Uniform Effluent Standard	
		Total Ⓣ	Whose Volume of Effluents Per Day is More Than 50 m³ ①	Less Than 50m³/d ②	Sum of ② Which Discharge Waste Containing Toxic Substances ③	①+③ =④	(%) ④/ΣⓉ
1	Mining and	103	62	41	3	65	0.2
1-2	Pig House, Horse House, Cow House Concerning Livestock Farming	33,718	498	33,220	0	498	1.7
2	Meat Packing and Poultry Processing	2,697	816	1,881	1	817	2.7
3	Sea Foods Manufacturing	9,640	976	8,664	0	976	3.1
4	Canned and Frozen Vegetables and Fruits Manufacturing	2,367	479	1,888	0	479	1.5
5	Miso, Soy-Sauce, Edible Aminoacid, Glutamic Acid Vegetable Sauces and Vinegar Manufacturing	3,527	176	3,351		176	0.7
6	Wheat Flour Manufacturing	45	13	32		13	0.0
7	Sugar Manufacturing	99	81	18		81	0.3
8	Bakery and Confectionary	1,582	97	1,485		97	0.3
9	Rice Cake and Malt Manufacturing	696	67	629		67	0.2
10	Soft Drink Manufacturing and Brewery	4,728	556	4,172	3	559	1,8
11	Feed Staff and Organic Fertilizering Manufacturing	485	163	322		163	0.5
12	Oil and Fat Manufacturing	334	107	227		107	0.3
13	Yeast Manufacturing	9	7	2		7	0.0
14	Starch Manufactuing	309	234	75		234	0.7
15	Dextrose Manufacturing	67	41	26		41	0.1
16	Noodles Manufacturing	4,390	91	4,299		91	0.3

Table 8 Classification of Specified Factories (in Fiscal 1974) 2/3

No. of the Table 1 in Cabinet Order	Kind of Business or Name of Facilities	Number of Specified Factories				Number of Specified Factories to be Controlled by the Uniform Effluent Standard	
		Total Ⓣ	Whose Volume of Effluents Per Day is More Than 50 m³ ①	Less Than 50m³/d ②	Sum of ② Which Discharge Waste Containing Toxic Substances ③	①+③ =④	(%) ④/Σ Ⓣ
17	Bean Foods Manufacturing	21,841	131	21,710		131	0.4
18	Instant Coffee Manufacturing	9	5	4		5	0.0
19	Textile Industry	4,934	1,615	3,319	108	1,723	5.4
20	Wool Scouring and Washing	63	36	27	4	40	0.1
21	Synthetic Textile Manufacturing	52	49	3		49	0.2
22	Chemical Finishing of Woods	162	10	152	46	56	0.2
23	Pulp and Paper Manufacturing	1,258	818	440		818	2.7
24	Fertilizer Manufacturing	73	56	17	1	57	0.2
25	Sodium Hydroxide and Potassium Hydroxide Manufacturing	23	23			23	0.1
26	Inorganic Pigment Manufacturing Inorganic Chemicals	69	44	25	3	47	0.1
27	Manufacturing Excluding Items of 25 and 26	405	259	146	19	278	0.9
28	Ethylene Derivates Manufacturing (Carbide Process)	69	35	34	3	38	0.1
29	Coal Tar Products Manufacturing	5	2	3		2	0.0
30	Fermentation Industry Excluding Items of 5, 10	39	22	17		22	0.1
31	Methane Derivates Manufacturing	18	12	6		12	0.0
32	Synthetic Plastic Manufacturing	67	38	29	1	39	0.1
33	Synthetic Plastic Manufacturing	271	196	75	3	199	0.6
34	Synthetic Rubber Manufacturing	23	15	8		15	0.0
35	Organic Gum Chemicals Manufacturing	9	9			9	0.0
36	Synthetic Detergent Manufacturing	24	19	5		19	0.0
37	Petrochemical Industries (Carbohydrate and Its Derivates)	123	103	20		103	0.3
38	Soap Manufacturing	26	7	19		7	0.0
39	Hydrogenated Oil Manufacturing	7	4	3		4	0.8
40	Fatty Acids Manufacturing	10	6	4		6	0.0
41	Perfumery Manufacturing	28	17	11		17	0.1
42	Gelatine and Glue Manufacturing	17	8	9		8	0.0
43	Photo Sensitive Goods Manufacturing	22	9	13	5	14	0.0
44	Natural Resin Manufacturing	7	3	4		3	0.0
45	Woods Chemical Manufacturing						

Table 8 Classification of Specified Factories (in Fiscal 1974) 3/3

No. of the Table 1 in Cabinet Order	Kind of Business or Name of Facilities	Number of Specified Factories				Number of Specified Factories to be Controlled by the Uniform Effluent Standard	
		Total Ⓣ	Whose Volume of Effluents Per Day is More Than 50 m³ ①	②	Sum of ② Which Discharge Waste Containing Toxic Substances ③	①+③ =④	(%) ④/ΣⓉ
46	Organic Chemical Manufacturing Excluding Items From 28 to 45	246	159	87	6	165	0.5
47	Pharmaceutical Manufacturing	221	139	82	5	144	0.5
48	Gunpowder Manufacturing	12	8	4		8	0.0
49	Pesticides Manufacturing	52	11	41	10	21	0.1
50	Reagent Manufacturing	6	2	4	1	3	0.0
51	Oil Refining Industry	81	63	18		63	0.2
52	Leather Manufacturing	462	161	301	10	171	0.5
53	Glass Manufacturing	698	107	591	357	464	1.5
54	Cement Manufacturing	2,889	163	2,726	150	313	1.0
55	Ready Mixed Concrete Manufacturing	2,999	277	2,722	130	407	1.3
56	Organic Snad Board Manufacturing	24	4	20		4	0.0
57	Synthetic Black Lead Electrode	18	17	1		17	0.1
58	Raw Pottery Materials Manufacturing	1,295	253	1,042	28	281	0.9
59	Macadam Quarring	732	336	396		336	1.1
60	Sand Collection	2,284	1,211	1,073		1,211	3.8
61	Iron Industry	695	307	388	7	314	1.0
62	Nonferreous Metals	236	122	114	15	137	0.4
63	Metaric Goods Manufacturing and Machinery Industry	1,385	570	815	232	802	2.5
64	Town Gas and Coke Manufacturing	162	137	25	4	141	0.4
65	Acid and Alkali Treatment Facilities of Metal Surface	4,594	1,609	2,985	481	2,090	6.6
66	(a) Electro-Plating Facilities	3,289	1,177	2,112	1,882	3,059	9.6
	(b) Hotel and Inn	61,948	3,473	58,475	1	3,474	10.9
67	Laundry (a) Washing Facilities	25,826	241	25,585	1	242	0.8
68	Photo Developing	759	238	521	267	550	1.6
69	(a) Slaughter House	512	278	234	1	279	0.9
70	Waste Oil Treatment Facilities	35	7	28		7	0.0
71	(a) Automatically Washing Facilities of Car	11,071	248	10,823		248	0.8
	(b) Natural Science Institute and Laboratory	3,744	744	3,000	1,980	2,724	8.5
72	Night Soil Treatment Plant (More Than 501 Population Served.)	6,607	5,482	1,125	2	5,484	17.2
73	Sewage Treatment Plant	413	410	3		410	1.3
74	Waste Water Treatment Plant	185	144	41	9	153	0.5
	Total	227,929	26,112	201,817	5,779	31,891	100.0

Table 9 Volumes of Factory Effluent by Scale

(Unit: 1,000 m^3/day)

Number of Employee	Number of Establishment	Sum of Industrial Shipment (billion yen)	Amount of Water Used ①	Water Used by Objective		Amount of Sewage ①-(②+③)
				For Boilers ②	For Materials ③	
Total	417,876	125,702	49,606	2,819	612	46,175
1~29	361,075	20,755	7,357	434	136	6,787
30~49	23,270	8,245	2,299	183	106	2,010
50~99	18,330	12,620	3,382	245	102	3,035
100~199	8,464	13,378	4,275	228	98	3,949
200~299	2,643	8,737	3,293	217	42	3,034
300~499	1,905	11,466	5,052	250	62	4,740
500~999	1,385	17,070	9,680	523	56	9,101
1,000 and over	804	33,431	14,268	739	10	13,519

Note: Amount of Industrial Waste Water from the establishments of 1 to 29 employee is estimated.

3.1.3 INSTALLATION STATE OF PRE-TREATMENT FACILITIES

The Sewerage Law before revision provided that the general manager of the public sewerage system, through the enactment of regulations, establish standards for the installation of pre-treatment facilities in accordance with the standards set by Cabinet Order. Therefore, the standards applied for the construction of pre-treatment facilities vary with cities, towns and villages. The number of establishments that are required to install pre-treatment facilities is 13,750. The number of those already equipped with such facilities is 8,665 or 63% of the total. The ratio of the factories which have already had pre-treatment facilities is higher among those having the facilities such as electroplating discharging harmful substance.

3.1.4 SURVEILLANCE ON FACTORY EFFLUENT

The general managers of the public sewerage system can order his staff to enter land or buildings owned by others and inspect sewage systems, pre-treatment facilities and other objects for the purpose of maintaining the structure and functions of the public sewerage system and conforming the water quality of the effluent to the technical standards provided for under Article 8 of the Law. The number of inspections conducted in fiscal 1975 totaled approximately 27,000.

For proper management of the public sewerage system, its general manager can also collect from the operator of the specified facilities reports on the state of factories discharging sewage and pre-treatment facilities as well as on the water quality of the sewage discharged by these facilities. The number of reports collected during fiscal 1975 totaled about 6,700.

The administrative actions taken by the general manager of the public sewerage system during fiscal 1975 under the Sewerage Law are as follow:

Order for Improvement:	67 cases
Recommendation for Improvement:	271 cases
Warning:	46 cases
Total:	384 cases

The figure represents an increase of 328 cases over fiscal 1974. This is a result of strengthened surveillance by the general manager of the public sewerage system just

Table 10 Types of Specified Factories

No. of the Table 1 in Cabinet Order	Kind of Business or Name of Facilities	Number of Specified Factories	Number of Required Pretreatment Facilities (A)	Number of Installed Pretreatment Facilities (B)	(B)/(A) %
1	Mining and Coal Washing	3	3	2	67
1-2	Pig House, Horse House, Cow House Concerning Livestock Farming	36	9	5	56
2	Meat Packing and Poultry Processing	278	106	53	50
3	Sea Foods Manufacturing	822	233	113	48
4	Canned and Frozen Vegetables and Fruits Manufacturing	587	99	44	44
5	Miso, Soy-Sauce, Edible Aminoacid, Glutamic Acid Vegetable Sauces and Vinegar Manufacturing	319	98	35	36
6	Wheat Flour Manufacturing	67	30	7	23
7	Sugar Manufacturing	35	8	4	50
8	Bakery and Confectionary	1,016	276	76	28
9	Rice Cake and Malt Manufacturing	246	45	17	36
10	Soft Drink Manufacturing and Brewery	506	187	105	56
11	Feed Staff and Organic Fertilizering Manufacturing	30	23	12	[illegible]
12	Oil and Fat Manufacturing	57	45	19	42
13	Yeast Manufacturing	3	1	1	100
14	Starch Manufacturing	26	10	6	60
15	Dextrose Manufacturing	13	10	9	90
16	Noodles Manufacturing	1,196	120	29	24
17	Bean Foods Manufacturing	3,425	292	78	27
18	Instant Coffee Manufacturing	2	1	1	100
19	Textile Industry	5,070	558	191	34
20	Wool Scouring and Washing	78	62	12	19
21	Synthetic Textile Manufacturing	1	1	1	100
22	Chemical Finishing of Woods	6	2	0	0
23	Pulp and Paper Manufacturing	55	42	32	76
24	Fertilizer Manufacturing	3	2	1	50
25	Sodium Hydroxide and Potassium Hydroxide Manufacturing	2	2	2	100
26	Inorganic Pigment Manufacturing Inorganic Chemicals	25	23	19	83
27	Manufacturing Excluding Items of 25 and 26	83	75	67	89
28	Ethylene Derivates Manufacturing (Carbide Process)	6	3	3	100
29	Coal Tar Products Manufacturing	2	2	2	100
30	Fermentation Industry Excluding Items of 5, 10	9	5	4	80
31	Methane Derivates Manufacturing	6	3	3	100
32	Synthetic Plastic Manufacturing	113	42	37	88
33	Synthetic Plastic Manufacturing	37	28	16	57
34	Synthetic Rubber Manufacturing	9	5	5	100
35	Organic Gum Chemicals Manufacturing	4	3	3	100
36	Synthetic Detergent Manufacturing	28	13	8	62
37	Petrochemical Industries (Carbohydrate and Its Derivates)	12	9	7	78
38	Soap Manufacturing	34	19	17	89

Table 10 (Continued)

No. of the Table 1 in Cabinet Order	Kind of Business or Name of Facilities	Number of Specified Factories	Number of Required Pretreatment Facilities	Number of Installed Pretreatment Facilities	(B)/(A) %
39	Hydrogenated Oil Manufacturing	4	4	4	100
40	Fatty Acids Manufacturing	9	8	7	88
41	Perfumery Manufacturing	10	8	7	88
42	Gelatine and Glue Manufacturing	3	3	1	33
43	Photo Sensitive Goods Manufacturing	29	25	20	80
44	Natural Resin Manufacturing	2	2	2	100
45	Woods Chemical Manufacturing	4	1	1	100
46	Organic Chemical Manufacturing Excluding Items From 28 to 45	105	83	73	88
47	Pharmaceutical Manufacturing	88	62	51	82
48	Gunpowder Manufacturing	4	3	1	33
49	Pesticides Manufacturing	6	5	5	100
50	Reagent Manufacturing	5	5	5	100
51	Oil Refining Industry	24	19	19	100
52	Leather Manufacturing	211	197	18	9
53	Glass Manufacturing	71	51	38	75
54	Cement Manufacturing	67	33	23	70
55	Ready Mixed Concrete Manufacturing	88	49	41	84
56	Organic Snad Board Manufacturing	4	3	3	100
57	Synthetic Black Lead Electrode	5	2	2	100
58	Raw Pottery Materials Manufacturing	14	12	10	83
59	Macadam Quarring	8	2	1	50
60	Sand Collection	7	3	2	67
61	Iron Industry	267	113	89	79
62	Nonferreous Metals	76	65	54	83
63	Metaric Goods Manufacturing and Machinery Industry	822	283	197	70
64	Town Gas and Coke Manufacturing	24	19	15	79
65	Acid and Alkali Treatment Facilities of Metal Surface	2,409	2,092	2,006	96
66	(a) Electro-Plating Facilities	2,327	2,295	1,964	86
	(b) Hotel and Inn	11,623	634	269	42
67	Laundry (a) Washing Facilities	8,273	666	119	18
68	Photo Developing	1,162	834	559	67
69	(a) Slaughter House	31	30	21	70
70	Waste Oil Treatment Facilities	94	77	70	91
71	(a) Automatically Washing Facilities of Car	2,650	1,304	1,031	79
	(b) Natural Science Institute and Labor Laboratory	565	232	116	50
72	Night Soil Treatment Plant (More Than 501 Population Served.)	55	20	19	95
73	Sewage Treatment Plant	–	–	–	
74	Waste Water Treatment Plant	15	12	7	58

as in the case of the inspection of water quality under the Sewerage Law which sharply increased from about 12,000 cases in fiscal 1974 to about 17,000 cases in fiscal 1975.

Thus, guidance and surveillance on pre-treatment facilities in cities, towns and villages are being strengthened in recent years with particular emphasis on those in cities. The number of officials engaged in these works in city, town and village offices totaled 733 during fiscal 1976.

Table 11 Surveillance on Pre-treatment Facilities (1975)

Classification	Municipalities	Number
Reporting Commencement of Utilization (Accumulation)	Major Cities (10)	12,107
	Others	13,532
	Total	25,639
Inspection of Private Sewer	Major Cities (10)	14,077
	Others	12,902
	Total	26,979
Collection of Reports	Major Cities (10)	2,855
	Others	3,889
	Total	6,744
Order for Improvement	Major Cities (10)	135
	Others	249
	Total	384

Major Cities: Tokyo, Osaka, Sapporo, Yokohama, Kawasaki,
Nagoya, Kyoto, Kobe, Kitakyushu, Fukuoka.

3.2 AID SYSTEM FOR CONSTRUCTION OF PRE-TREATMENT FACILITIES

Under the Sewerage Law before revision, factories and other establishments were already obliged to equip themselves with pre-treatment facilities. As a result of the introduction of the prior checking system and the direct penalty system under the revised law, the construction of the pre-treatment facilities has become indispensable. The standards for the construction of the pre-treatment facilities are not uniform, since they are decided by regulations instituted respectively by cities, towns and villages. The factories which have not installed the pre-treatment facilities total 5,085 in number and represent 37% of those which are required such facilities. Among reasons for delayed action are lack of funds, lack of land, under planning and under construction as listed on Table 12. Many of the factories which have not yet built the pre-treatment facilities are small businesses, and therefore the state and local governments have established aid systems to promote the construction of the facilities as listed on Table 13.

3.3 PUBLICATION OF GUIDANCE MANUAL

Under the revised Sewerage Law, cities, towns and villages serving as general manager of the public sewerage system are obliged to assume clerical works concerning the prior checking system and other procedures. This means that cities, towns and villages will handle the same clerical works as those assumed by the prefectural governments (partly delegated to major cities) provided for under the

Water Pollution Control Law, including the prior checking system. In order to help smaller administrative bodies carry out the newly-assigned work smoothly, the state has decided to publish a guiding manual for the prior checking system and other procedures, and the Sewerage Association is now studying the plan.

The state has also instruct the local governments to prepare new registers for the pre-treatment facilities in a unified form with a view to strengthening surveillance on specified establishments and giving proper guidance.

Table 12 Reasons for Non-Installing Pre-treatment Facilities

	Specified Factories (%)	Non-specified Factories (%)
Lack of Money	34.3	24.3
Lack of Space	17.8	0.9
Planning to Install	15.0	5.7
Tentative Treatment	11.1	26.3
Planning to Move	0.8	0.1
Others	21.0	42.7
Total	100.0	100.0

Table 13.1 Aid System of Financial Institution for Installing Pre-treatment Facilities (March 1976)

Corporation	Object Enterprise	Maximum Sum of Loan	Interest Rate (per year)	Term of Redemption	Amount of Loan (1976 F.Y.)
Environmental Pollution Control Service Corporation	Big Enterprise Small-to-Middle Enterprise (Capital Stock; less than 100 million yen Employee; less than 300)	Non-limit	Big Enterprise Cooperative Anti-Pollution Facilities First 3 years; 7.5% Later; 7.7% Individual Anti-Pollution Facilities; 7.7% Small-to-Middle Enterprise Cooperative Anti-Pollution Facilities First 3 years; 4.5% Later; 5.0% Individual Anti-Pollution Facilities; 6.0%	Cooperative Anti-Pollution Facilities Instrument and Equipment; 10 years (unredeemed 1 year) Others; 20 years (unredeemed 3 years) Individual Anti-Pollution Facilities; 10 years (unredeemed 1 year)	Cooperative Anti-Pollution Facilities, Individual Anti-Pollution Facilities, Others; 127 billion yen
Japan Development Bank (A recommendation of the authority concerned is necessary to a loan.)	Enterprises Except Object Enterprises of the Loan of the Smaller Business Finance Corporation	Non-limit	First 3 years; 7.7% Later; 8.2%	About 10 years	Pollution Prevention Facilities, Anti-Pollution Facilities, Others; 128 billion yen
Smaller Business Finance Corporation	Small-to-Middle Enterprise (Capital Stock; less than 100 million yen Employee; less than 300)	Direct Loan: 150 million yen Loan by Agent 40 million yen	First 3 years; 7.0% Later; 7.2%	10 years (unredeemed 2 years)	Safety Loan, Anti-Pollution Loan; 53 billion yen
People's Finance Corporation	Mainly Smaller-to-Middle Enterprise (Capital Stock; less than 10 million yen Employee; less than 100)	18 million yen	First 3 years; 7% Later; 7.2%	10 years (unredeemed 2 years)	Safety Loan, Anti-Pollution Loan; 10 billion yen
Smaller Business Promotion Corporation	Cooperative Anti-Pollution Business of Business Cooperative Association etc.	Non-limit	Non-interest	15 years (unredeemed 2 years)	2 billion yen

Table 13.2 Aid System of Major 7 Cities for Installing Pre-treatment Facilities

City	Name of System	Maximum Sum of Loan	Interest Rate	Term of Redemption	Amount of Loan (1976 F.Y.)
Tokyo Metropolis (All 23 ward have the system to supply money for interest other than these.)	Tokyo Metropolis Mediation System for Loaning of Anti-Pollution Fund	20 million yen	Metropolis Supplies Money for Interest so that Interest Rate may be 2 percentage.	7 years (unredeemed 6 months)	5 billion yen (including fund for movement)
	Tokyo Metropolis Loaning System for Anti-Pollution Fund	7 million yen	2%	7 years (unredeemed 6 months)	300 million yen
Yokohama	Yokohama City Loaning System for Small-to-Middle Enterprise (Anti-Pollution Fund)	25 million yen	City Supplies All Money for Interest.	3 to 9 years	600 million yen
Nagoya	Nagoya City Mediation System for Loaning of Anti-Pollution Facilities Improvement Fund	25 million yen	6.8%	7 years (unredeemed 1 year)	2 billion yen
Kyoto	Kyoto City Anti-Pollution Fund System	20 million yen	1.5%	12 years	660 million yen (including fund for movement)
Osaka	Osaka City Loaning System for Anti-Pollution Facilities	20 million yen	City Supplies Money for Interest so that Interest Rate may be 2 percentage (1 percentage for small enterprises)	7 years	150 million yen
Kobe	Kobe City General Loaning System of Anti-Pollution Fund	20 million yen	City Supplies Money for Interest so that Interest Rate may be 2.76 percentage.	7 years	650 million yen
Kitakyushu City	Kitakyushu City Loaning System of Anti-Pollution Fund	10 million yen	City Supplies All Money for Interest.	7 years	300 million yen

Table 13.3 Utilization State of Aid System for Installing Pre-treatment Facilities (1971~1975 F.Y.)

Classification	System	Number
National Government	Environmental Pollution Control Service Corporation	45
	People's Finance Corporation	327
	Smaller Business Finance Corporation	56
	Smaller Business Facility Modernization Fund	14
	Environmental Sanitation Business Finance Corporation	1
	Sub-total	443
Local Government	Prefecture	117
	Municipality	834
	Sub-total	951
Total		1,391

Mentioned above are a summary of the revision of the Sewerage Law, the state of controls on factory effluent before the revision and the new system of surveillance under the revised law. A report on the state of the execution of the revised law will be made on the next occasion.

RECENT TOPICS IN WATER POLLUTION CONTROL IN JAPAN

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RECENT TOPICS IN WATER POLLUTION CONTROL IN JAPAN

1. CURRENT PROGRESS ON WATER POLLUTION CONTROL IN JAPAN

The state of this country's water pollution caused by cadmium and other toxic substances has improved lately with the years. In terms of BOD (or COD in the case of lakes and sea water), the principal index of water quality with regard to organic substances, water pollution in major water bodies has definitely slowed – even improved in certain areas – since 1969, thanks to the rigorous regulation of effluents throughout the country in recent years. However, there still remain areas where water pollution is not in good condition, and they include the river waters that run through cities where the population and industries are heavily concentrated and the coastal waters, particularly those of bays and inland seas where the waters are trapped within a certain area. In the following pages, we will raise some questions to be discussed at the 5th US/JAPAN conference on sewage treatment technology.

Table 1 Degree of Nonconformity with Toxic Substance Environmental Standards

Substance	Fiscal Year	Percentage	Substance	Fiscal Year	Percentage
Cadmium	1970	2.8 %	Arsenic	1970	1.0 %
	71	0.7		71	0.4
	72	0.34		72	0.29
	73	0.32		73	0.31
	74	0.37		74	0.27
	75	0.31		75	0.24
Cyanides	70	1.5	Total Mercury	70	1.0
	71	1.2		71	0.3
	72	0.5		72	0.04
	73	0.2		73	0.01
	74	0.06		74	See Notes*
	75	0.02		75	
Organic Phosphorus	70	0.2	Alkyl Mercury	70	0
	71	0.2		71	0
	72	0		72	0
	73	0		73	0
	74	0		74	0
	75	0		75	0
Lead	70	2.7	Total	70	1.4
	71	1.4		71	0.6
	72	0.7		72	0.3
	73	0.65		73	0.23
	74	0.37		74	0.20 **
	75	0.32		75	0.17
Hexavalent Chrome	70	0.8			
	71	0.1			
	72	0.07			
	73	0.08			
	74	0.03			
	75	0.02			

* Total Mercury Fiscal '74 and '75 $\frac{\text{Numbers of Stations Exceeding Standards}}{\text{Total Number of Stations}} = 0\%$

** Except Total Mercury

Table 2 Degree of Nonconformity with Standards Relating to the Living Environment

(FY '75, ( ) shows FY '74)

Body of Water Category		Numbers of Samples Not Meeting Standards / Total Numbers of Samples (%)	
Rivers	AA	22.2	(24.2)
	A	22.6	(23.0)
	B	21.3	(23.3)
	C	17.4	(18.7)
	D	12.5	(13.4)
	E	24.1	(27.1)
	Total	21.3	(22.4)
Lakes and Marshes	AA	32.5	(33.4)
	A	39.1	(34.7)
	B	46.2	(48.1)
	C	30.3	(25.3)
	Total	38.4	(35.1)
Sea Areas	A	18.1	(18.8)
	B	14.2	(14.0)
	C	8.4	(7.1)
	Total	16.1	(16.0)

2. SURVEYS FOR TOTAL EMISSION CONTROL FOR ORGANIC POLLUTANTS

The regulation introduced under the Water Pollution Control Law has only taken into account the concentration of pollutants, but it seems insufficient that to meet the demand for some specific water areas such as lakes and bays.

Therefore, surveys for introduction of so-called "total emission control," i.e., limitation of the emission of certain pollutants at each individual source so that the total emission volume for a given water area is less than a level to meet the environmental standards of the water area concerned.

The item for total emission control in water pollution control is intended to be COD, one of the items relating to living environment, while the items for total emission control in air pollution control are the items relating to human health.

In May 1975, Water Quality Bureau established a study committee for total emission control (headed by Professor Ishibashi of Tokyo University). Since then, the committee has studied fundamental structures of total emission control, i.e., pollutants or items to be controlled, water areas to be controlled (that means to be designated), the method of calculation of total emission volume, allocation method of total emission volume for each individual source, and the method of supervision and measurement. The committee, after five times sessions, on September 1975, issued an interim report consists of three parts (I. Fundamental plan, II. Way of regulation, III. Supervision and measurement).

According to this, a fundamental survey is now carrying on as to Tokyo Bay, Ise Bay and Seto Inland Sea for the introduction of total emission control of pollutants, FY 1976. Also, in FY 1977, similar survey is intended as to Lake Biwa, Lake Suwa and Lake Kasumi.

3. COUNTER MEASURES FOR EUTROPHICATION

Among the recent features of water pollution, eutrophication problems in the specific water areas, such as Seto Inland Sea, Tokyo Bay and Lake Kasumi, are most troublesome.

As to "red tides" in the Seto Inland Sea, although the incidence of formation has been increasing annually, the percentage of cases where damage has been suffered has been decreasing (Table 3). Part of the increased occurrence is actually due to more comprehensive surveillance and reporting and not to more frequent red tide formation, but even taking this factor into account, the virtually irreversible process of eutrophication is still clearly being accelerated in those areas where nutrient salts using microorganisms as their medium are building up in the water. The result is the abnormal phenomenon of red tide formation and other types of secondary organic pollution.

Table 3 Change in the Incidence of Red Tides

Year / Item	1967	1968	1969	1970	1971	1972	1973	1974
No. of Time (A)	48	61	67	79	136	164	210	298
No. of Times with Fishery Damage (B)	8	12	18	35	39	23	18	17
$\frac{(B)}{(A)}$ x 100 (%)	17	20	27	44	29	14	9	6

Source: Fishery Agency

Hence, from FY 1972, surveys of phosphorus and nitrogen are carried out about major sea areas and lakes. From FY 1975, in Seto Inland Sea and Ise Bay, and in FY 1977, adding Tokyo Bay, surveys for phosphorus and nitrogen are carried out, as to input charge of each individual source, dissolving from bottom sediment, plankton and so on.

As to synthetic detergent, JIS (Japan Industrial Standard) relating to synthetic detergent for clothes is amended in December 1976, and phosphate content is reduced from 8 to 20% to less than 12%.

4. OTHER WATER POLLUTION PHENOMENA

4.1 EFFLUENT DISCHARGED IN WARM TEMPERATURE

Recently, power stations are becoming much larger and also they are apt to be constructed in the areas where there have been no water pollution problems before. So it is worried about that the effluent discharged from those power stations in state of warm temperature affect badly to marine life and fishery.

In December 1975, "Thermal Pollution Committee" of Water Quality Committee of The Central Council for Control of Environmental Pollution issued an interim report as to thermal pollution. It says that, the necessity of continuing surveys, and sufficient environmental impact assessment when a new power station is planned, and also the necessity of monitoring of environment after the completion of construction.

4.2 LONG TIME DURATION OF TURBID WATER FLOW FROM DAMS

One of the problems recently raised is the long time duration of turbid water flow down from dams after rainfall. Generally speaking, Japanese rivers are usually clear in turbid because of geological and geographical conditions, and when it rains the turbidity of river flow will go up promptly and when it clear up the turbidity of river flow go down again promptly. But, if a dam is constructed it is usual there comes a long time duration of comparably high turbid water after rain is up. And this gives a bad effect to utilization of river water and fishery. And also is the problem that a dam brings a cold water according to the way of discharge.

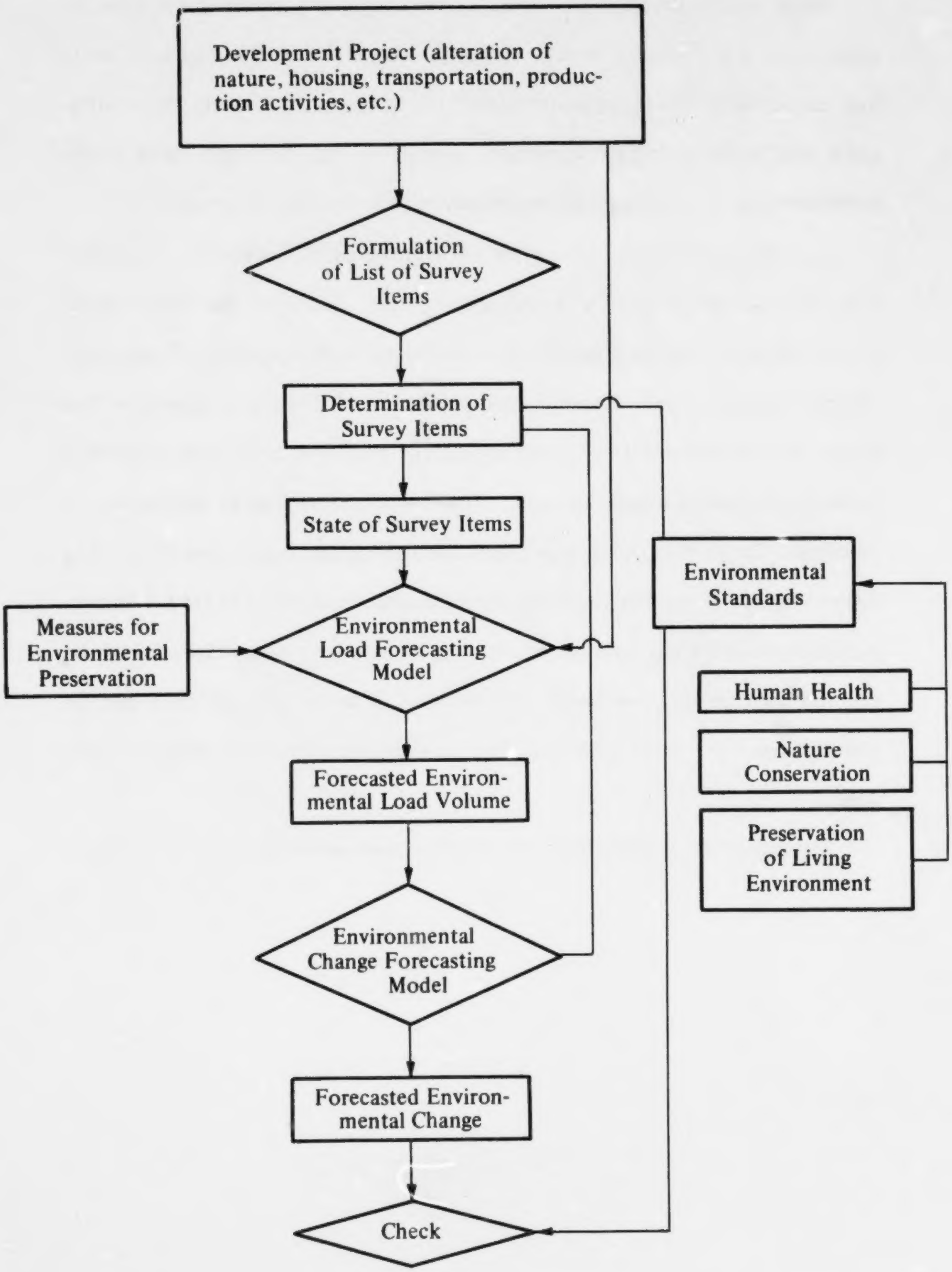
5. ENVIRONMENT IMPACT ASSESSMENT

When various development works in relation to any public water areas are carried out, it is necessary to take proper preventive measures so that such works may not adversely affect the environment. For this purpose, it is very important to make environmental impact assessment, thereby to make a careful check of the possible effects such development works may have on the adjacent environment.

The above-mentioned basic policy was approved at the Cabinet meeting held in June 1972 and was adopted as a government policy. Thereafter, the environmental impact assessment has been carried on in accordance with this policy. This concept of environmental impact assessment was embodied in various laws enacted at 71st session of the National Diet during 1973. The provision in the Seto Inland-Sea Conservation Law is a good example. This law stipulates that an application for permission for building specific industrial facilities in the coastal areas of the Seto Inland-Sea shall be accomplished with the statement of the results of the preliminary evaluation made by the surveys on possible influences the proposed industrial facilities may have on the surrounding environments. The law also stipulates that the said document should be made available to public inspection for a period of three weeks.

The general procedures for environmental impact assessment is shown in Fig. 1.

Fig. 1 Procedures for Environmental Impact Assessment


Development Project (alteration of nature, housing, transportation, production activities, etc.)
Formulation of List of Survey Items
Determination of Survey Items
State of Survey Items
Measures for Environmental Preservation
Environmental Load Forecasting Model
Forecasted Environmental Load Volume
Environmental Change Forecasting Model
Forecasted Environmental Change
Check
Environmental Standards
Human Health
Nature Conservation
Preservation of Living Environment

FIFTH US/JAPAN CONFERENCE
ON
SEWAGE TREATMENT TECHNOLOGY
PAPER NO. 2

STUDIES ON STORM AND COMBINED SEWER OVERFLOW

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MINISTRY OF CONSTRUCTION
JAPANESE GOVERNMENT

STUDIES ON STORM AND COMBINED SEWER OVERFLOW

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K. Takeishi, Ministry of Construction

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CHAPTER 1. WATER QUALITY CHARACTERISTICS OF STORM & COMBINED SEWER OVERFLOW

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1. WATER QUALITY CHARACTERISTICS OF STORM & COMBINED SEWER OVERFLOW

1.1 CURRENT STATUS OF THE COMBINED SEWER SYSTEM IN JAPAN

1.1.1 COMBINED SEWER SYSTEM

In Japan as of 1975, 583 municipalities had been practicing the sewage works. In 1976, 65 new municipalities are expected to enter upon sewage works.

The population served by sewers in 1975 is estimated to have been 22.8% of the total population.

In Japan, those cities which have a long experience in the sewage works have the combined sewer system. According to a 1972 survey conducted by the Ministry of Construction, the combined sewer system accounted for 73% of the total sewered area, covering 69% of the total sewered population.

As the problems of the combined sewer overflow come to the fore, the number of cities adopting the separate sewer systems is on a steady rise in recent years.

The Ministry of Construction is also a strong promotor of the separate sewer systems. Table 1.1 shows the changes in the past three years of the number of municipalities by type of sewerage.

The many municipalities which have combined sewer system adopt the separate sewer system in new sewered areas, so the share of the combined sewer system is decreasing gradually. Nevertheless, the significance of the combined sewer overflow on the contribution of pollution loads in waters has not yet been reduced at all. In large cities where the sewerage is making a large step toward improvement through secondary treatment, the contribution for water pollution by combined sewer overflow will be increased more and more.

Table 1.1 Relative Use of Combined Sewers of Local Governments

Year / Sewer system	1973	1974	1975
Combined sewer	70	52	47
Hybrid	158	176	185
Separate sewer	217	287	351
Total	445	515	583

1.1.2 ON-GOING MEASURES AND RESEARCH ACTIVITIES FOR THE COMBINED SEWER SYSTEMS

1.1.2.1 MEASURES FOR THE COMBINED SEWER SYSTEM

In Japan, little has been reported of examples taking radical measures against pollution caused by combined sewer overflow. Most of municipalities have been bending their efforts for the expansion of sewer-served areas. Although they have full knowledges on the problems the combined sewer system has, their efforts don't reach to the measures against them.

In the major cities where the sewerage system has long been used, improvement of overflow chamber and enlargement of sewer capacity are being pushed forward in order to recoup shortage of the capacity of the existing sewer system and to overcome the problem of reduced dilution ratio of wet-weather overflow. These improvements, however, are still in the stage of local scale practice, and come nowhere near providing a meaningful way against the water pollution by wet-weather overflow.

What is standing in the way of changing the combined system to the separate system lies in the vast sums of money required, narrow width of road in densely populated area, and the fact that storm water from storm sewer itself is polluted. Converting to the separate system is not considered to be the best solution.

Most of measures taken or being planned to control the water pollution by combined sewer overflow are either detention or sedimentation of wet-weather overflow.

A facility installed in Osaka is the only one example of this type now we can see in Japan, except for two additional facilities which are now under construction.

All these three are touched upon briefly in the next section. Some new districts which adopted combined sewer systems install storm water sedimentation basins in their sewage treatment plants in order to overcome the problems of wet-weather overflow.

1.1.2.2 Current Research Activities

For the purpose of studying the improvement measures for the combined sewer system, the Ministry of Construction established an research organization named "Research Committee on Sewer System Problems." The members of the Committee consist of the engineers from the Ministry of Construction, the Japan Sewage Works Agency, major municipalities and representative medium-sized municipalities. Of these municipalities, eleven adopt the combined sewer system, and one the separate sewer system.

The principal objective of this Committee is to investigate about the combined sewer system in each municipality by standardized manners, exchange findings and formulate practical improvement measures.

The Committee is scheduled to work from 1975 to 1980. For the first three years, the Committee will bend their energies to collect data with emphasis on the fact-finding survey.

The data obtained this way is significant not only for the improvement of the combined sewer system, but also as a basis for the planning of future sewer system.

Section 3 summarizes the findings acquired by the Committee in 1975.

The Urban River Section of the Public Works Institute, Ministry of Construction, developed a water quality simulation model of the combined sewer system. The mode which stands on the concept of the retention of pollution load gives the pollutegraph from a given hydrograph.

It is corroborated by application of this model to some drainage areas that the simulated results agree well with actual measurements. The model is considered to become a useful tool in assessing the improvement of combined sewer system because it can estimate the pollutegraph from the hydrograph calculated by the modified RRL method from the precipitation record.

The results of this study are discussed in the next chapter.

1.2 EXAMPLES OF MEASURES AGAINST COMBINED SEWER OVERFLOW PROBLEMS

In Japan, there are not so many practical examples of measures to control water pollution due to combined sewer overflow.

Briefed here are three examples in Osaka and Yokohama.

1.2.1 STORM WATER SEDIMENTATION TANK IN OSAKA

In 1975, the Osaka Municipal Government constructed storm water sedimentation tank consisting of two basins by taking advantage of the improvement work of Nakanoshima Pumping Station at the center of the city. The dimension of each basin is 3.5 m in width, 20.2 m in length and 4.5 m to 5.0 m in depth. In the future, four additional basins will be constructed. When all these tanks are completed, the storage capacity will become 2,000 m^3 or worth 4.4 mm of rainfalls in the area. This facility is the only one example in Japan of measures against wet-weather overflow.

1.2.2 STORM WATER DETENTION TRUNK SEWER IN OSAKA

In Osaka, innundation frequently visits lowlands because of marginal use of catch basin and subsidence due to excessive use of ground water. In order to solve this problem, installation of a new interceptor was planned and has been under way since 1973.

Its major portion having an inside diameter of 6,000 mm will be installed 25 m deep with the length of about 3 km, and will provide a storage capacity of some 80,000 m^3 or worth 6.6 mm of rainfalls in the area.

This trunk sewer is planned to serve for the control of water pollution due to wet-weather overflow without detriment to the original purpose of innundation control.

1.2.3 STORM WATER SEDIMENTATION TANK IN YOKOHAMA

The Yokohama Municipal Government is in the process of constructing storm water sedimentation tank consisting of eighteen 6.0 m wide by 35 m long by 6.0 m deep basins with a total capacity of 22,680 m^3 equivalent to 6.4 mm of rainfalls in the area.

The tanks are of the underground type, and their covered top is planned to be an athletic ground open to the public. These tanks are expected to reduce the pollution loads by 10% or more in SS and almost the same degree in BOD as compared with the separation device.

1.3 INTERIM RESULTS OF BASIC SURVEYS BY THE RESEACH COMMITTEE ON COMBINED SEWER SYSTEM PROBLEMS

1.3.1 METHOD OF SURVEY

For the purpose of collecting extensive data on the characteristics of wet-weather combined sewage, twelve drainage areas different in drainage system and land use were selected as the study areas in the eleven member municipalities. The designated areas have already been perfectly covered by the combined sewer system, have a sizable tract each, and also are favoured with conditions permitting hydraulic and hydrologic analysis with ease.

In one city which has adopted the separate sewer system, a survey area was also sited to fulfil similar conditions. A survey station was installed at the downmost end of the sewer in each survey area for flow measurement and sampling. At several spots in the survey area, rainfall gaging was also carried out. The sampled water was subjected to analysis according to analysis according to the unified methods.

In 1975, flow measurement and sampling were conducted one to two times in dry-weather and two to four times in wet-weather in the combined sewer survey areas and four times in wet-weather in the separate sewer survey area. In the separate sewer municipality, the survey area was relocated in 1976 to a suburb where three-times surveys were carried out in wet-weather.

In addition, geographical and social conditions, including land use patterns and demographic status, were also investigated in each survey area.

1.3.2 OUTLINE OF SURVEY AREAS

The combined sewer survey areas are outlined in Table 1.2 (a), and the separate sewer survey areas in Table 1.2 (b).

The following is a list of survey areas and a brief explanation of each.

Combined sewer survey areas

A: Single-family residential area apart from the city center
B: Populated urban area with a mixed consist of residential houses and stores
C: Area with a mixed consist of small to medium factories, residential houses and stores
D: Area with residential zones and shopping quarters
E: Residential area on a terrace bordering on an unban area
F: Area with low-story and medium-story shopping quarters and amusement quarters
G: Area with low-story shopping quarters, amusement quarters and medium-story business quarters
H: Dense area packed with low-story residential houses, stores and factories mainly of textile and dyeing
I: Typical high-story civic and business center
J: Area sited on a spit in the estuary, with residential quarters, and commercial quarters, wholesale markets and small factories along a trunk road
K: Suburban residential area on a terrace with many company-owned residential

Table 1.2 (a) Summary of Surveyed District (Combined Sewer System)

Drainage district	Drainage system	Drainage area (ha)	Population		Sewer diameter (mm)	Max. flow rate in dry-weather (m^3/s)	Land use						Road area		Impervious area	
			Daytime	Resident			Residential (ha)	(%)	Commercial (ha)	(%)	Industrial (ha)	(%)	(ha)	(%)	(ha)	(%)
A	Pumping	44.33	6,655	4,016	300~ 900	0.057	44.33	100	0	0	0	0	6.95	15.7	15.27	34.4
B	Gravity flow	540.6	165,019	155,091	□ 6000 × 4325	2.11	397.5	73.5	68.29	12.6	74.80	13.9	116.6	21.6	288.0	53.3
C	Pumping	269.5	58,690	61,167	□ 4000 × 3600	1.57	16.79	6.2	46.84	17.4	205.87	76.4	3.82	1.4	186.3	69.1
D	Pumping	35.13	12,298	9,003	□ 1650 × 1650	0.10	21.45	61.1	13.68	38.9	0	0	4.46	12.7	16.08	45.8
E	Gravity flow	22.09	3,579	4,270	250~ 1200	0.052	21.38	96.8	0.71	3.2	0	0	2.39	10.8	7.68	34.8
F	Gravity flow	68.37	38,268	8,358	230~ □ 1950 × 1950	0.282	0	0	68.37	100	0	0	21.05	30.8	55.43	81.1
G	Gravity flow	39.5	12,800	1,780	250~ 1650	0.189	0	0	39.5	100	0	0	9.80	24.8	31.02	78.5
H	Grayity flow	148.49	37,180	25,280	250~ □ 3000 × 1930	0.809	32.10	21.6	32.86	22.1	83.53	56.3	31.27	21.1	114.46	77.1
I	Pumping	45.50	43,200	804	300~ □ 2000 × 1350	0.385	0	0	45.50	100	0	0	24.3	53.4	39.1	85.9
J	Pumping	215.14	27,000	31,727	250~ □ 2200 × 2200	0.261	108.54	50.5	41.6	19.3	65.0	30.2	44.77	20.8	127.95	59.5
K	Grayity flow	57.6	3,973	3,863	200~ □ 2100 × 1680	0.028	57.6	100	0	0	0	0	7.0	12.2	15.0	26.0
L	Gravity flow	17.61	9,700 (day) 17,000 (night)	1,098	230~ □ 1360 × 1060	0.247	0	0	17.61	100	0	0	5.35	30.4	16.40	93.1

Table 1.2 (b) Summary of Surveyed District (Separate Sewer System)

Drainage district	Drainage system	Drainage area (ha)	Population		Sewer diameter (mm)	Land use						Road area		Impervious area	
			Daytime	Resident		Residential (ha)	Residential (%)	Commercial (ha)	Commercial (%)	Industrial (ha)	Industrial (%)	(ha)	(%)	(ha)	(%)
M	Gravity flow	17.17	9,300	2,695	□1200× 1030~ □30×70	8.55	49.8	8.02	46.7	0.60	3.5	5.72	33.3	15.7	91.4
O	Gravity flow	26.75	2,420	3,227	□2500× 1800	26.75	100	0	0	0	0	4.99	18.7	21.26	79.5

houses

L: Typical amusement quarters on a spit in the estuary, enticing extremely large night population compared with daytime one

Separate sewer survey areas

M: Tier on the sea, with business quarters, commercial quarters, hotels, public facilities, government institutions, multi-family residential houses

O: Newly developed residential area on a suburban terrace, with single-family houses

1.3.3 SURVEY RESULTS

1.3.3.1 Water Quality Characteristics of Combined Sewer

Summarized in the following are the results of dry- and wet-weather water quality survey made in 1975 of the twelve survey areas in the eleven municipalities in different localities in Japan.

(a) Findings of Dry-weather Survey

In each survey area, dry-weather around-the-clock survey was carried out once or twice. During the survey, sewage flow was measured and once-every-30 minutes sampling were conducted for water quality analysis.

Table 1.3 shows dry-weather average water qualities and concentration ranges in respective survey areas.

Followings are the water quality characteristics of respective survey areas in dry-weather and the factor that are considered attributable to them.

C: Industrial quarters account for 75% of total area, discharging high concentrations of heavy metals.

E: Residential, but high in BOD, COD and SS.

H: A good number of textile and dyeing factories in the area resulting high BOD, COD and SS.

I: Typical business quarters discharging weak effluents having a soluble-to-total BOD ratio of 14.4%.

K: Low BOD; quantities of pipeline desposits are suspected to be.

L: Densely built amusement quarters generating a large volume of sewage for the area; high BOD.

While these areas have different characteristics, they also have something in common with each other as follows.

Table 1.3 Dry Weather Average Concentration of Combined Sewage

Drainage district	Date	Dry Weather Arerage Concentration (Upper: Average, Lower: Range) mg/ℓ										
		BOD	S-BOD	COD	SS	VSS	T-N	T-P	Zn	Cu	Pb	Coliform group $\times 10^4$/mℓ
A	Oct. 22 ~23	136 21~560	79.9 19~468	37.2 6.2~158	55.0 14~163	40.0 3~128	15.8 ND~40.6	4.14 0.11~18.8	0.02 ND~0.08	0.017 ND~0.58	ND	2.91 0.05~14
B	Jul. 15	103 23~151	–	–	83.2 12~157	–	–	–	–	–	–	–
C	Sep. 21 ~22	127 12~229	42.1 5.8~72.9	53.0 10.0~136	88.3 12~205	60.9 3~123	14.7 5.17~27.7	4.31 0.94~8.24	2.23 0.16~9.13	0.844 0.02~10.2	0.04 0.01~0.09	–
D	Aug. 26 ~27	110 35~281	82.0 17~198	60.0 24~191	84.8 19~258	60.8 7~230	21.5 12~52	2.05 1.0~5.5	ND	ND	ND	80 20~280
E	Dec. 3	134 16~374	49.0 4~184	117 15~293	202 8~844	129 8~388	21.3 10.6~60.1	5.20 1.6~12.5	0.30 0.07~0.65	0.03 ND~0.08	0.006 ND~0.05	5.18 0.5~26
F	Sep. 2 ~3	84.8 34.7~147	31.6 9.9~53.6	41.1 11.9~67	76.0 26~184	51.9 26~90	12.4 5.49~21.1	4.13 1.4~8.7	0.25 0.15~0.47	0.057 0.02~0.14	0.01 ND~0.03	6.4 1.2~9.7
	Nov. 11 ~12	122 39.5~180	36.4 10.1~63.2	35.6 23.2~56.1	79.0 27~132	54.2 21~104	13.8 6.8~28.8	4.06 1.6~7.6	0.15 0.11~0.22	0.04 0.01~0.07	0.01 ND~0.03	6.5 2.8~9.2
G	Aug. 27 ~28	93.2 7.6~205	47.6 4.4~132	38.9 8.7~63	64.0 13~125	53.9 10~104	11.5 3.0~18.0	1.46 0.29~2.20	0.19 0.09~0.40	0.037 ND~0.15	ND	77 21~260
H	Oct. 23 ~24	321 66~691	208 44.3~311	232 35~359	120 22~194	86.3 19~129	15.3 9.56~29.7	5.04 2.08~8.20	0.663 0.10~1.51	0.079 0.02~0.16	0.01 ND~0.04	2.4 0.011~11
I	Jan. 12 ~13	101 11~180	14.5 4.0~29.3	45.6 16.3~75.7	95.7 19~202	60.4 14~109	17.9 1.60~30.7	2.57 1.01~3.42	0.12 ND~0.20	0.05 ND~0.07	ND	17 1.5~40
J	Oct. 31	116 14~203	68.4 10.6~152	74.2 17.3~148	78.2 12~187	57.0 10~140	21.5 11.2~47.4	10.6 2.7~23.1	1.5 0.05~27	ND	ND	5.7 1.6~20
K	Oct. 1	51.7 6.3~123	18.5 4.5~43.0	32.5 7.8~79.5	55.8 10~145	46.5 6~120	11.0 4.1~35.0	3.08 1.13~7.32	0.08 ND~0.6	0.013 0.007~0.025	ND	11 0.52~38
L	Aug. 4 ~5	143 21.4~297	56.3 10.7~109	47.9 14~80.4	107 35~540	52.0 15~214	15.2 4.32~26.4	2.27 0.95~3.24	0.18 0.09~0.49	0.02 ND~0.16	ND	80 1.1~820
	Feb. 2	180 31~494	–	70.1 15.8~150	100 18~203	–	22.0 5.97~32.5	3.13 0.85~5.13	–	–	–	–

a. VSS/SS is in the range of 0.63 to 0.88, except for one survey area.

b. Three survey areas (F, G, and L) show flow rate 10 to 30% less in winter than in summer, and an increase in BOD of 30 to 40% on the average in winter over summer.

c. No definite differences by land use are noticed

Table 1.4 shows daily flow and loadings per capita discharged. In Japan, it has been generally accepted as a sewage works planning practice that the sanitary sewage volume and BOD load per day per person are about 500 lit. and 60 g, respectively. It is noteworthy that even in typical residential areas such as A and E, these standard values are exceeded by a large margin.

Table 1.4 Summary of Per Capita Runoff Loadings

(Per capita per day)

District	Popula-tion*	Flow (ℓ)	BOD (g)	SS (g)	K-N (g)	T-P (g)	
A	5,335	738	100	40.7	11.7	3.05	
B	160,055	661	68.1	55.0	–	–	
C	59,929	1,583	202	140	23.3	6.82	
D	10,650	536	59.9	46.6	11.8	1.13	
E	3,924	702	93.6	140	–	3.78	
F	23,313	711	60.4	55.3	9.09	3.05	(Sep. 2~3)
		510	62.3	40.5	7.21	2.15	(Nov. 11~12)
G	7,290	882	82.2	56.2	10.2	1.29	
H	31,230	1,155	367	140	18.1	5.88	
I	22,002	429	43.4	41.1	7.68	1.10	
J	29,363	487	56.6	38.6	10.5	5.35	
K	3,918	356	18.3	19.7	3.94	1.09	
L	9,266	1,352	192	145	20.5	3.06	(Aug. 4~5)
		1,258	226	126	–	3.93	(Feb. 2)

* Average of Daytime and Resident Population Shown in Table 1.2 (a).

(b) Findings of Wet-weather Survey

Wet-weather survey was conducted two times to four times in each survey area.

i) Wet-weather runoff and water characteristics

Fig. 1.1 shows the relationship between total rainfall and total runoff. The runoff coefficient is largely influenced by the impervious area ratio. As shown in Table 1.2, some survey areas have a large impervious area ratio of more than 90%. In these areas, the runoff coefficient is high. Taken altogether, however, the runoff coefficient lies in the range of 40 to 80%.

Table 1.5 shows average characteristics and concentrations of combined sewage in wet-weather conditions.

Figs. 1.2 (a) through (d) show wet-weather BOD, SS, T-P, and K-N in terms of

Table 1.5 Summary of Average Quality of Combined Sewage in Wet Weather

District	Date		Rainfall (mm)	Wet Weather Average Concentration (Upper: Average, Lower: Range) mg/ℓ										
				BOD	S-BOD	COD	SS	VSS	K-N	T-P	Zn	Cu	Pb	Coliform Group $\times 10^4$/mℓ
A	1	Nov. 15 ~16	26.5	57.0 10~265	37.1 6~152	40.8 18~90	115 44~325	44.0 12~236	8.61 1.82~19.9	2.68 0.07~7.46	0.069 0.04~0.18	ND	ND	1.6 0.1~6.5
	2	Nov. 27	1.0	181 90~405	92.3 33~234	112 66~146	440 152~604	175 86~282	22.2 14.0~30.8	8.00 1.99~26.4	0.20 0.10~0.38	0.04 ND~0.05	0.05 ND~0.07	3.2 0.3~28
B	1	Jul. 10	6.5	120 76~186	–	–	117 75~174	–	–	–	–	–	–	–
	2	Jul. 12	8.3	106 76~186	–	–	208 68~580	–	–	–	–	–	–	–
C	1	Sept. 23 ~24	52.0	63.3 7.2~347	12.7 2.7~68.4	51.1 9.5~304	170 28~1250	113 11~691	9.77 2.23~38.3	2.18 0.16~11.8	0.971 ND~13.4	0.320 ND~7.70	0.049 ND~0.27	–
	2	Sept. 29	36.0	66.4 14.1~145	–	–	85.5 10~287	–	–	–	–	–	–	–
	3	Nov. 14 ~15	9.1	99.7 35~190	31.9 15.7~59.3	60.6 28.1~105	148 42~385	112 12~265	11.4 6.12~21.4	4.15 1.55~8.56	0.930 0.36~1.69	0.341 0.09~1.13	0.064 0.02~0.15	–
D	1	Oct. 3	2.25	83.2 30~182	60.0 24~120	43.7 24~68	129 56~240	88.0 38~200	19.0 9~35	3.52 1.5~6.4	0.20 0.1~0.3	ND	ND	18 4~50
	2	Nov. 19	18.50	44.0 13~155	15.0 7~45	42.6 24~126	118 49~228	106 47~220	6.03 2~20	1.40 0.5~3.1	0.064 0.01~0.14	0.010 ND~0.11	ND	41 19~76
	3	Feb. 5	11.07	53.6 34~128	30.0 24~60	36.7 28~60	130 83~183	40.0 11~68	8.07 5~12	0.74 0.4~2.2	0.38 0.18~0.58	0.061 0.02~0.12	ND	2.9 1.2~13
E	1	Oct. 24	6.5	89.8 25~329	29.7 12~68	51.8 23~186	207 25~915	80.6 17~515	12.3 6.0~27.7	4.30 2.9~11.6	0.46 0.13~1.76	0.016 ND~0.12	0.047 ND~0.16	3.64 0.52~12
	2	Feb. 5 ~6	29.0	65.3 13~503	24.8 5~173	62.0 16~228	168 12~1090	71.0 9~614	12.8 4.1~101	3.10 1.1~14.9	0.45 0.13~1.54	0.044 ND~0.21	0.061 ND~0.37	3.2 0.2~54.6
	3	Feb. 16	33.0	34.5 8~279	7.50 2~105	40.1 9~268	209 4~630	66.0 4~343	7.27 0.8~36.3	3.15 0.9~10.1	0.47 0.12~1.88	0.066 0.02~0.49	0.038 ND~0.24	1.51 ND~56
F	1	Oct. 7	85.8	26.4 11.1~81.5	14.4 5.9~43.7	13.0 4.4~42.2	90.0 24~520	44.2 10~182	5.52 2.66~15.9	1.43 0.20~5.25	0.20 0.11~0.42	0.02 0.01~0.05	0.01 ND~0.04	1.5 0.1~6.6
	2	Oct. 24	23.1	27.8 6.8~124	11.0 2.8~52.3	13.1 3.6~45.8	26.9 8~185	17.8 7~65	4.04 2.02~15.1	1.18 0.12~5.21	0.18 0.12~0.32	0.138 ND~1.01	0.004 ND~0.02	1.3 0.5~6.8
	3	Nov. 7	40.1	27.1 9.2~247	12.8 6.1~45.8	9.8 5.8~45.8	84.9 9~565	54.4 7~220	5.25 2.08~16.6	1.64 0.11~5.21	0.12 0.04~0.18	0.01 ND~0.04	0.0056 ND~0.11	1.3 0.8~5.2
G	1	Aug. 22	2.5	83.4 34~288	–	36.9 21.2~81.3	133 56~323	–	–	–	–	–	–	–
	2	Sep. 8	5.0	127 59.5~250	33.3 23.0~67.7	75.0 31.6~153	214 53~550	101 37~194	9.18 5.6~14.0	1.59 0.9~2.9	0.51 0.18~0.77	0.065 0.02~0.11	0.088 ND~0.18	49 20~79
	3	Sep. 18	2.5	109 70.2~170	39.3 25.5~60.4	65.2 36.4~113	161 52~392	128 59~242	16.8 9.5~25.7	1.83 1.3~2.8	0.41 0.31~1.34	0.077 0.04~0.12	0.043 0.03~0.11	41 5.9~140
	4	Oct. 7	5.0	147 97.3~256	33.2 20.2~74.9	95.7 48.8~146	357 74~718	211 91~376	2.93 5.4~12.0	2.33 1.6~3.1	0.57 0.21~0.74	0.06 0.03~0.10	0.075 ND~0.12	67 16~280
H	1	Jul. 7	15.5	134 58.9~365	67.9 28.0~190	93.2 47.5~237	192 41~571	104 34~323	8.16 4.41~17.8	1.15 0.27~3.42	0.97 0.20~2.23	0.19 0.06~0.43	0.033 ND~0.11	7.0 ND~23
	2	Aug. 6	24.0	117 56.4~349	70.8 23.1~213	88.1 35.6~265	203 39~668	93.1 25~328	10.1 4.13~28.6	1.16 0.30~2.40	1.44 0.41~4.60	0.17 0.05~0.48	0.02 ND~0.06	1.5 0.005~12
	3	Feb. 5	14.2	205 52.3~522	88.3 36.0~187	104 18.9~212	202 14~572	138 16~372	16.4 8.90~29.7	2.97 2.07~5.98	1.60 0.12~3.90	0.14 0.02~0.29	0.01 ND~0.05	2.9 0.25~21
I	1	Oct. 28 ~29	19.5	47.4 8.1~260	8.75 2.0~60.6	28.6 9.3~63.5	57.2 13.2~368	32.0 7.5~225	9.51 3.4~26.1	1.26 0.57~3.82	0.17 ND~0.50	0.081 ND~0.35	ND	4.3 0.20~40
	2	Nov. 6 ~7	36.0	31.0 2.6~163	3.84 1.0~15.9	26.6 5.9~81.0	134 19.8~256	43.3 7.6~136	5.12 1.2~17.0	1.12 0.10~3.56	0.29 ND~0.58	0.093 0.03~0.17	ND	0.78 0.034~6.7
	3	Nov. 27	8.0	58.4 31.4~121	9.92 6.2~17.0	49.7 39.6~77.7	121 67~355	73.7 46~129	18.9 10.1~26.0	2.95 2.30~4.04	0.33 0.13~0.57	0.14 0.08~0.20	ND	5.4 1.7~15
J	1	Feb. 5	1.0	170 100~314	91.9 76.9~118	131 91.4~167	173 70~294	94.2 41~145	22.0 19.2~25.3	4.70 3.13~9.37	1.16 0.11~1.58	0.14 ND~0.36	ND	4.0 1.2~7.7
	2	Feb. 18	18.0	21.7 10.3~92.4	8.74 5.4~29.7	30.6 18.2~104	106 32~177	42.3 10~86	4.00 2.5~12.9	0.66 0.40~1.82	0.335 0.173~0.596	0.01 ND~0.32	ND	1.4 0.40~9.9
	3	Mar. 11 ~12	11.0	51.1 10.1~146	20.5 6.2~77.9	43.3 13.5~93.3	92.2 25~210	56.0 13~161	9.20 3.4~23.2	1.54 0.55~3.65	0.40 0.25~0.61	ND	ND	3.87 0.44~9.7
K	1	Oct. 7	12.5	114 26~840	13.7 5~26	51.1 12.9~231	238 21~969	115 13~706	7.84 2.2~37.3	1.99 0.45~16.3	0.11 ND~1.1	0.028 0.009~0.101	0.02 ND~0.06	17 1.4~180
	2	Nov. 13	9.5	139 35~300	17.1 11.3~32.3	80.6 21.3~280	309 42~2130	182 24~1490	19.9 7.45~61.1	2.70 0.63~9.61	0.30 0.05~0.79	0.045 0.009~0.122	0.02 ND~0.06	26 2.0~120
L	1	Nov. 18	6.0	91.5 56~157	22.1 13.2~46.5	81.4 45.0~198	884 176~2850	167 55~510	16.1 11.1~22.9	3.04 1.15~5.15	0.22 ND~0.45	0.01 ND~0.10	ND	13 2.4~54
	2	Feb. 5	2.8	75.7 23.1~1791	29.7 10.8~89.0	45.6 20.1~72.8	73.3 23~137	53.1 22~99	14.2 6.8~20.1	2.10 0.59~3.59	0.411 ND~1.24	0.03 ND~0.08	ND	24 1.4~190
	3	Feb. 17	27.0	107 9.5~242	53.2 4.9~144	67.2 8.0~97.2	125 20~374	73.0 14~242	10.1 2.01~42.0	1.69 0.30~5.07	0.29 0.09~0.53	0.029 ND~0.16	ND	35 0.010~250

Fig. 1.1 Total Runoff and Rainfall

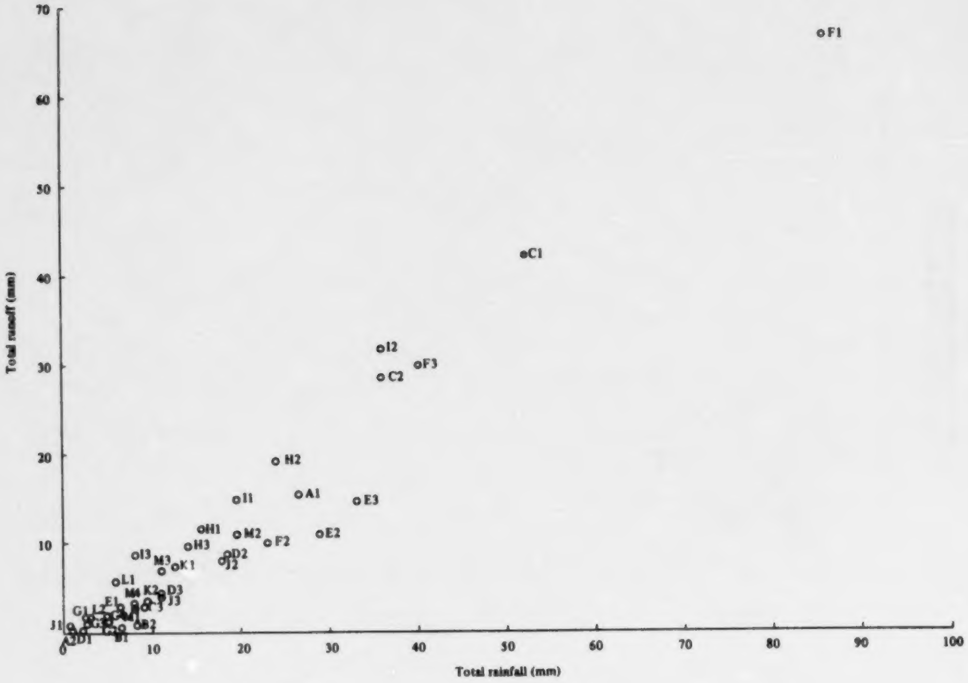


Fig. 1.2 (a) Discharge Volume and Average Concentration (BOD)

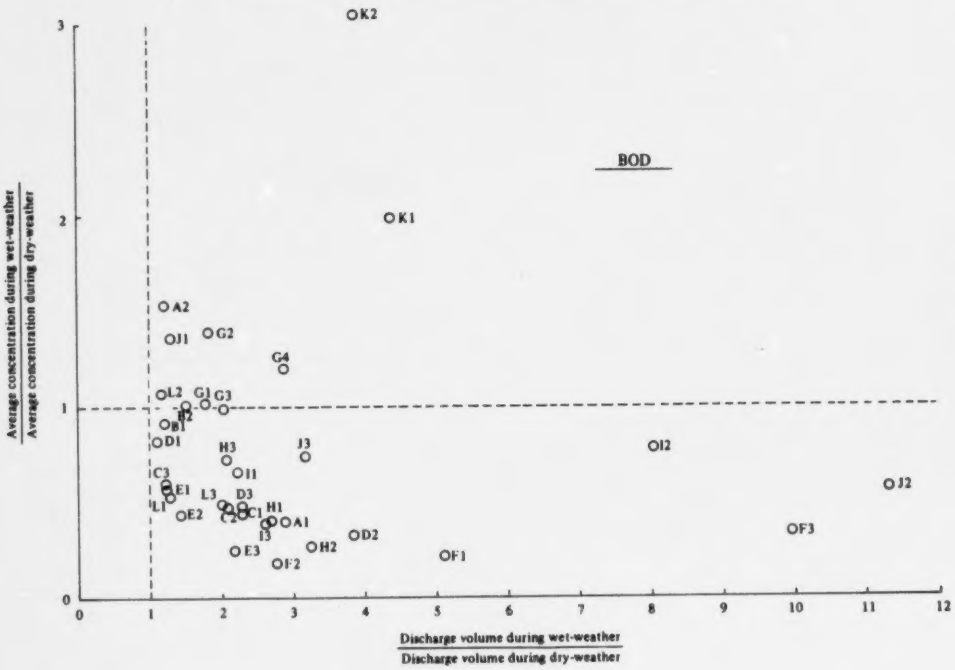


Fig. 1.2 (b) Discharge Volume and Average Concentration (SS)

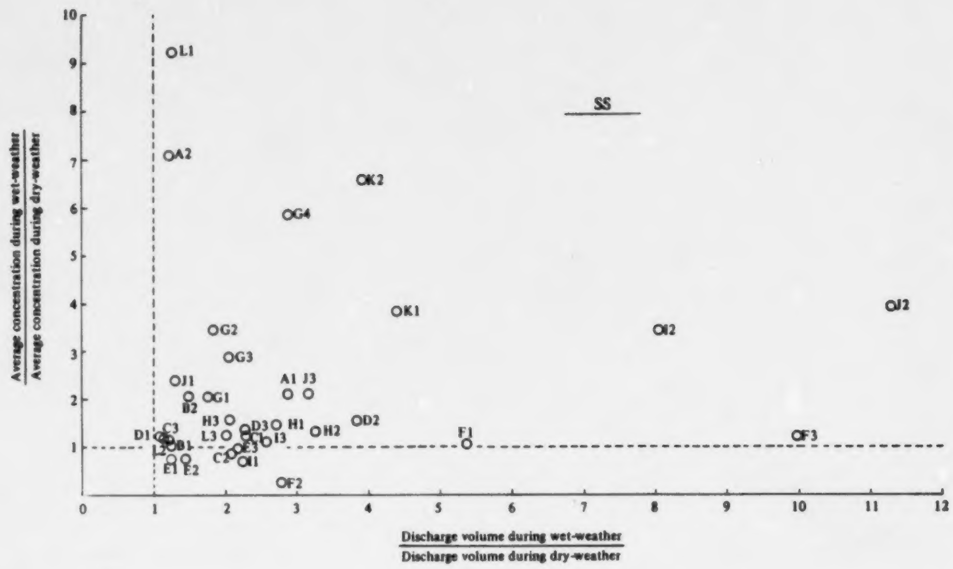


Fig. 1.2 (c) Discharge Volume and Average Concentration (Total-P)

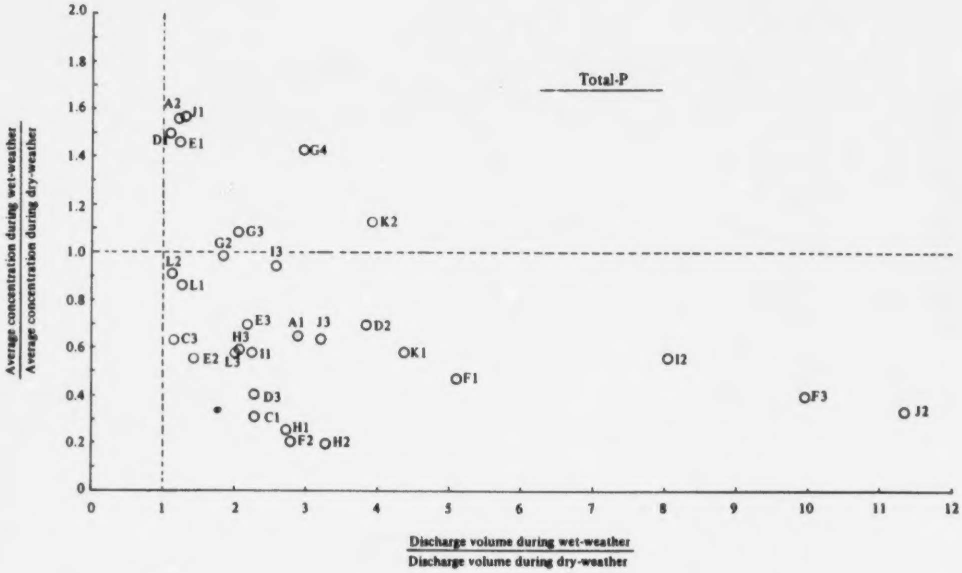
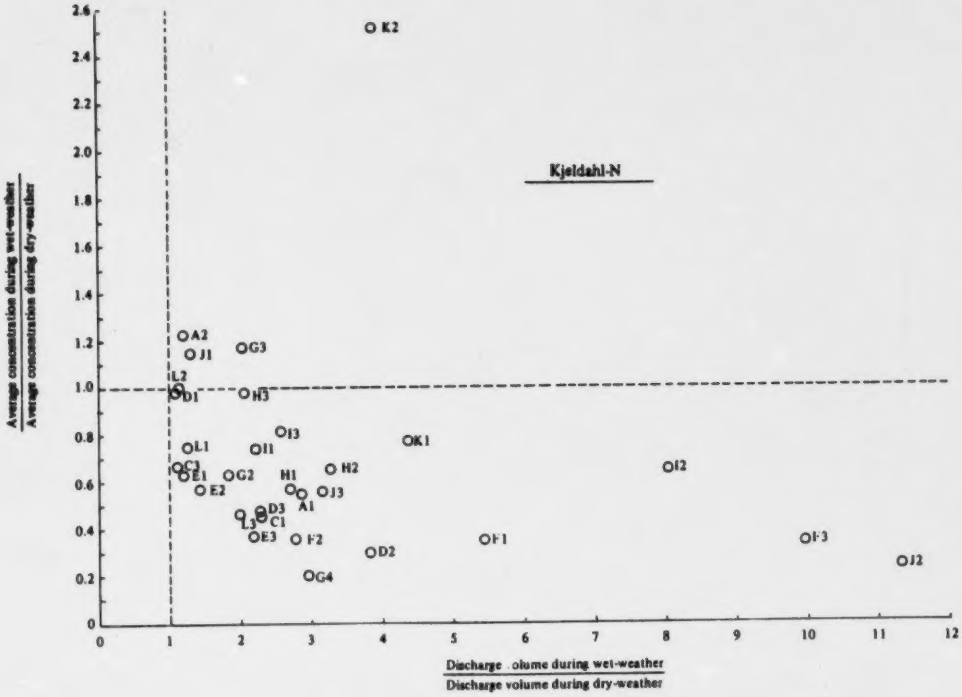


Fig. 1.2 (d) Discharge Volume and Average Concentration (K-N)



ratios to those in dry-weather. After a light rainfall, BOD, T-P and K-N become by some chance by far larger in concentration than in dry-weather. In case of a large scale storm, however, the concentrations are reduced by dilution effect and the loadings are generally held within several times those in dry-weather. As regard SS, however, the reduction of concentration is far less than others even in case of a large-scale rainfall. Namely, it remains almost the same as in dry-weather. This shows that the bulk of SS is conveyed into the sewer from the ground surfaces with unlimited sources of SS supply.

Fig. 1.3 shows the relationships between maximum storm water flow rates and runoff loadings of BOD and SS.

Although the runoff loadings of BOD and SS are different by an order or so among the survey areas, it is found that they have a significant relationship with flow rate.

As is clear from above, the mean values of most of water qualities can be said to decrease with increase in the scale of rainfall. However, the time change in runoff loadings is quite complicated, and defies rendition by mean qualities only.

Most of wet-weather records give the so-called "first flush."

Fig. 1.4 refers to the record in the survey area E in which the cumulative flow and cumulative loadings are shown in percentage.

It is highly suggestive of a large loading runoff in the early stage of rain runoff.

It should be noted however that compared with BOD and SS, the loading

Fig. 1.3 (a) Maximum Flow Rate and BOD Loadings Discharged

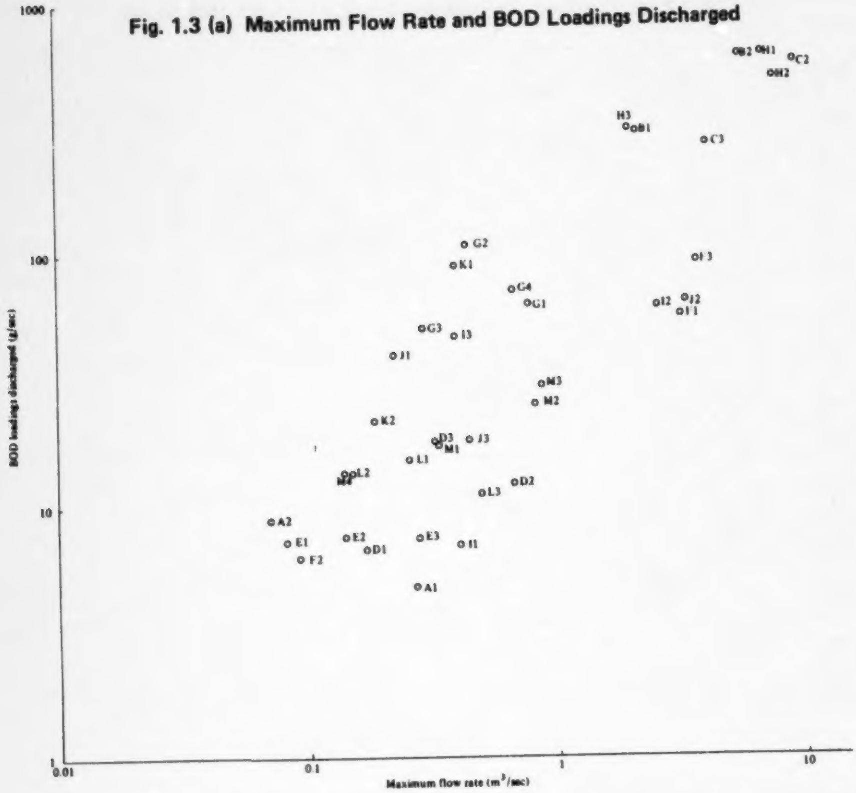


Fig. 1.3 (b) Maximum Flow Rate and SS Loadings Discharged

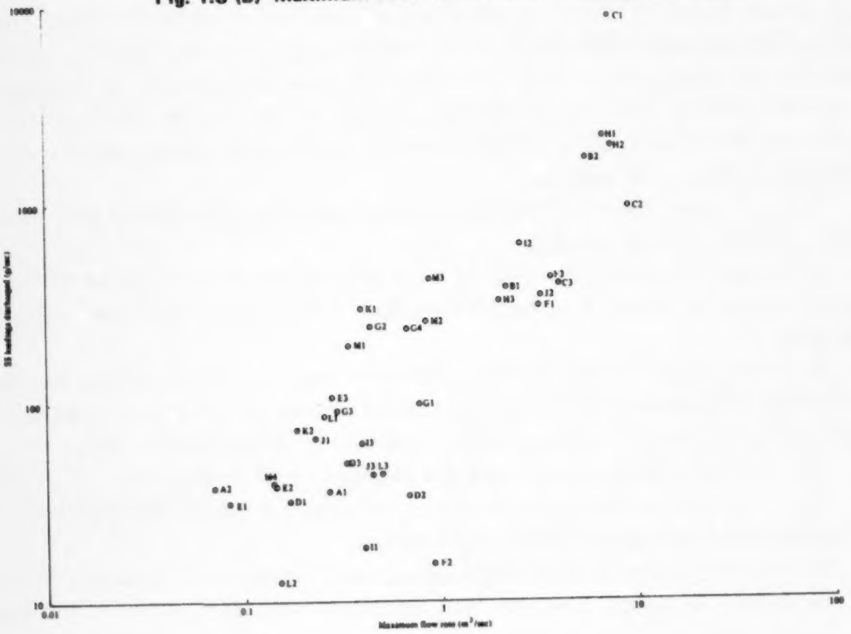


Fig. 1.4 Cumulative Flow Volume and Pollution Loadings Discharged

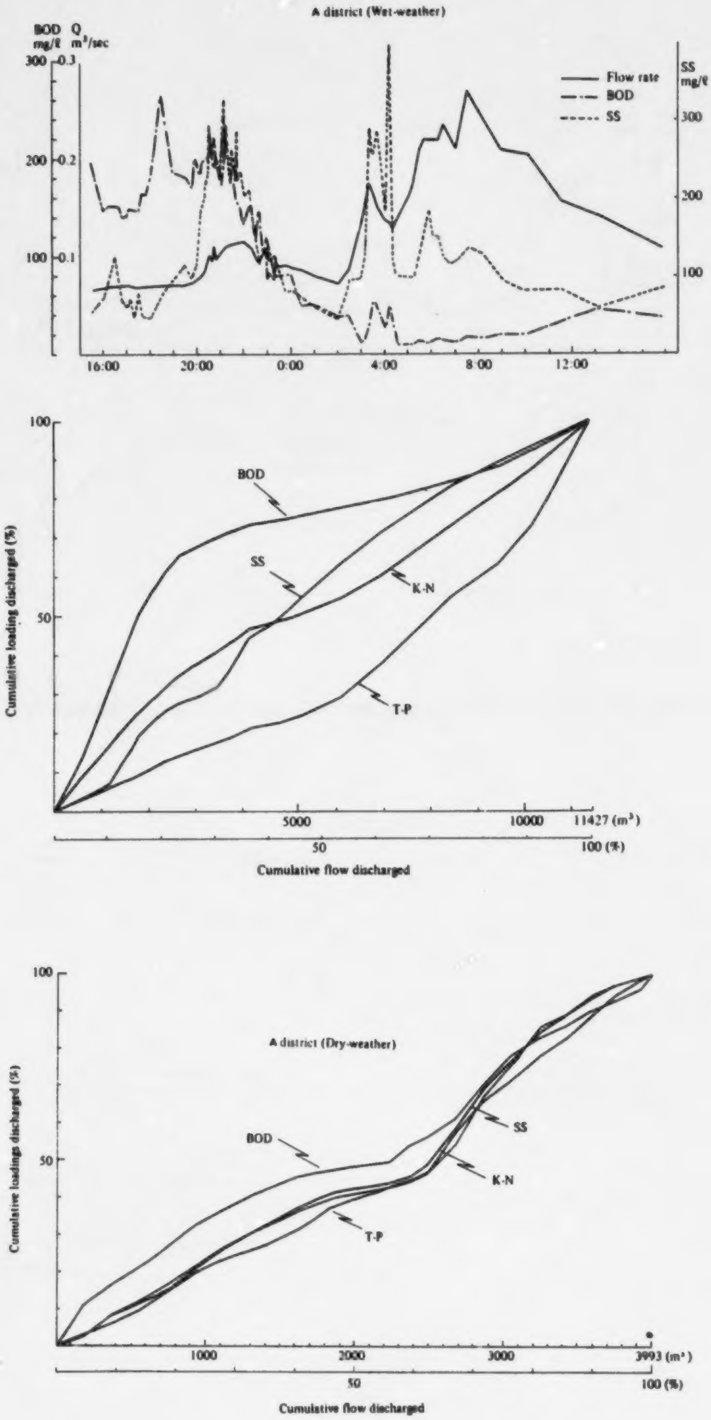


Fig. 1.5 (a) Variation of Water Quality in Combined Sewage (Dry-Weather)

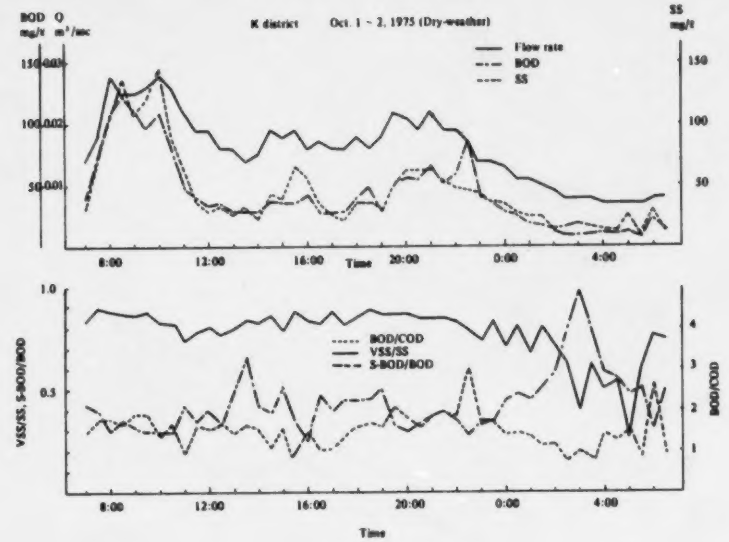
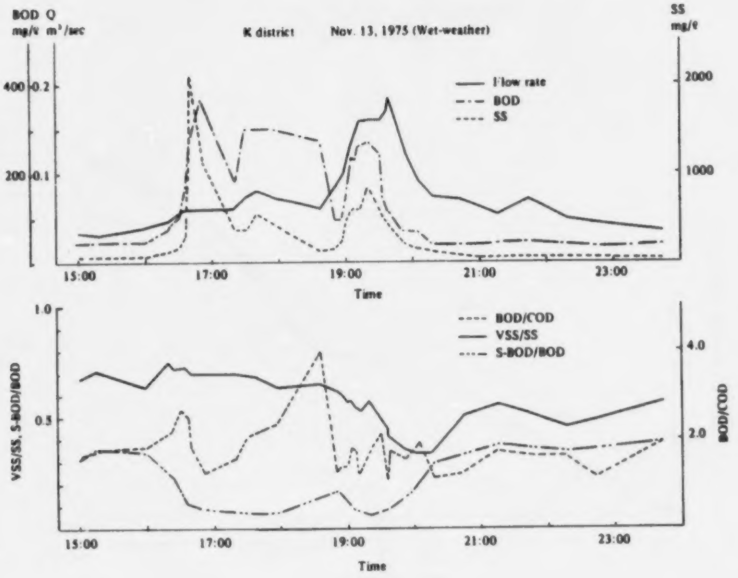


Fig. 1.5 (b) Variation of Water Quality in Combined Sewage (Wet-Weather)



runoff curves of T-P and K-N do not show so large a change with time, and this tendency is seen in most of rainfall observations. Hence it is inferred that the detention of wet-weather overflow by the storage tank in order to reduce pollution of waters will be ineffective in the removal of T-P and K-N as compared with BOD and SS.

Figs. 1.5 (a) and (b) show changes in water quality indices in the survey area K during dry- and wet-weather, respectively. With reference to dry-weather, BOD/COD, VSS/SS, S-BOD/BOD all are nearly constant in the daytime.

In the nighttime when the flow declines, VSS/SS decreases while S-BOD/BOD increases. On the other hand, BOD/COD does not show so conspicuous change. In wet-weather, every index shows a large change: in the rising period of runoff, S-BOD/BOD declines; in the first peak of SS runoff, VSS/SS shows no change, but in the second peak of SS when the sewage runoff has attained a maximum, VSS/SS falls largely.

The decrease in VSS/SS is considered due to supply of SS from ground surfaces.

With attention paid to the fact that VSS/SS is almost constant in dry-weather while it changes in wet-weather, a trial is made to divide SS runoff loading into ground-contributed component and the sewer deposit-contributed component which is supplied during dry-weather.

From the urban storm water discharge survey shown in 1.3.3.2, the ground-contributed SS is estimated at 0.2 in terms of VSS/SS.

The results of computation are shown in Fig. 1.6.

The first peak showing "first flush" is chiefly contributed by SS runoff from the deposits in the sewer. In the second peak, however, SS is divided almost equally between the deposit component and ground component. With reference to the rainfall shown in Fig. 1.6, the ultimate SS discharge of 674 kg is broken down as follows.

408 kg (61%) by sewer deposit runoff; 240 kg (36%) by ground supply; and 26 kg (3%) by sewage. Namely, the sewer deposit is found most responsible for SS discharge.

ii) Local characteristics

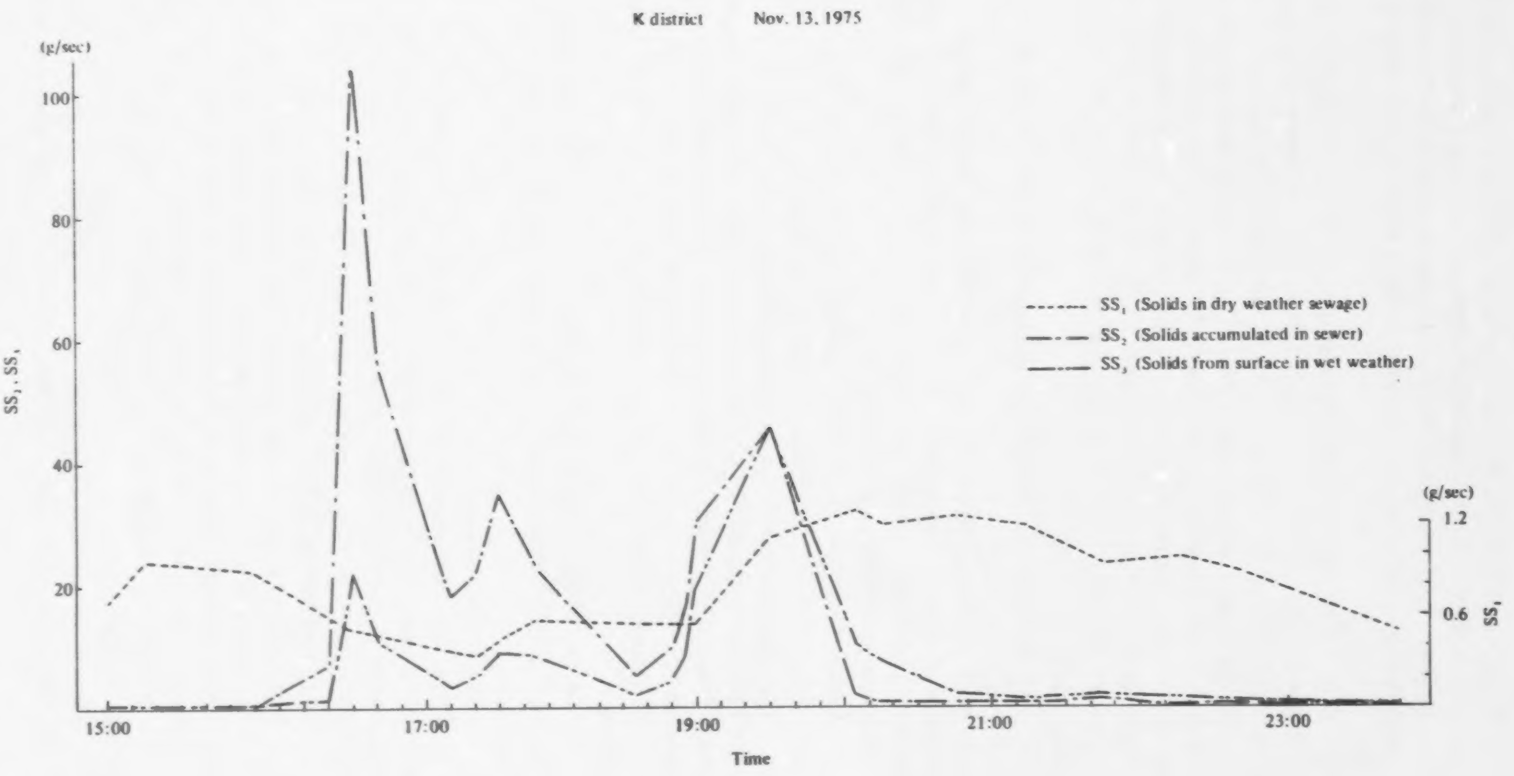
From the viewpoint of locality, what shows the most highest change in wet-weather as compared with dry-weather is the survey area K. In the second rainfall observation in the survey area K, for example, the average concentration is 2.7 times in BOD as much as that at the same hours in dry-weather, 5.6 times in SS as much, and the runoff loading is 10.5 times and 21.8 times, respectively.

The maximum concentration is 840 mg/ℓ for BOD, and 2,129 mg/ℓ for SS.

As shown in 1.3.3.1, the dry-weather concentrations in the survey area K are very small compared with the average concentrations in other survey areas. It is therefore conjectured that a considerable amount of loadings is deposited in the sewer during dry-weather and is flushed out in wet-weather to develop a salient increase in the loadings.

The survey area K is situated on a steep with a ground slope of 24.5‰. Considering this high slope, it is very queer why such deposit is developed in the

Fig. 1.6 Comparison of Solids in Wet Weather Sewage by Sources.



sewer. The clarification of the causes is left to further study in the future.

In contrast to the survey area K, the survey area E which forms also residential quarters on a steep slope of 20.4‰ shows only a bit increase in the wet-weather runoff loadings. In the first rainfall observation in survey area K, for example, the average concentration is 0.78 times in BOD and 1.02 times in SS as much as that in the dry-weather and the runoff loadings also remain small.

As explained in 1.3.3.1, it is worth noting that in the survey area K, SS, COD, etc. are considerably high in dry-weather sewage.

For all that the survey areas K and E are almost the same in land use and geographical conditions, they present quite different phases both in dry- and wet-weather pollutant discharges. In the survey area K, it is no doubt that there is something promoting deposition in the sewer.

It is therefore hoped that the causes be clarified to reflect in the engineering of sewage pipeline system.

iii) Correlation between water characteristics

Figs. 1.7 (a), (b) and (c) show the correlation between the characteristics obtained by the analysis of both dry- and wet-weather surveys. Here are some interesting findings.

In many survey areas, the ratio VSS/SS becomes smaller in wet-weather than in dry-weather, and the inorganic suspended solids increase in wet-weather.

From the correlation between BOD and TOC, it is understood that the refractory organics increases in wet weather.

This is very important in dealing with the treatment of wet-weather combined sewer overflow.

Of the heavy metals, zinc is found in every survey area. In many survey areas, it is found that zinc has a significant correlation with SS, though no substantial difference is seen between dry- and wet-weather.

1.3.3.2 Characteristics of Urban Storm Water Discharge

Aside from the water pollution due to combined sewer overflow, the urban storm water discharge has come to be more and more recognized as a major contributor of water pollution.

In the separate sewer municipality, member of the Research Committee on Combined Sewer System Problems, a survey area was established for investigating urban storm water discharge. Survey stations were set at the storm sewers in the survey area M forming commercial quarters in the center of city in 1975 and in the survey area O forming newly developed suburban residential quarters in 1976, for the purpose of wet-weather storm water sampling and discharge observation.

(a) Local Characteristics

Table 1.6 shows the mean characteristics and concentration ranges of urban storm water discharges observed at the two survey areas. In the survey area M where amusement quarters exist, the water quality is very poor; the concentration on the average of four rainfall observations were 38.9 mg/ℓ for BOD, 288 mg/ℓ for SS. The maximum values were 309 mg/ℓ and 1,180 mg/ℓ, respectively, suggesting that the urban storm water discharge itself is considerably contaminated.

Fig. 1.7 Correlation between Water Qualities

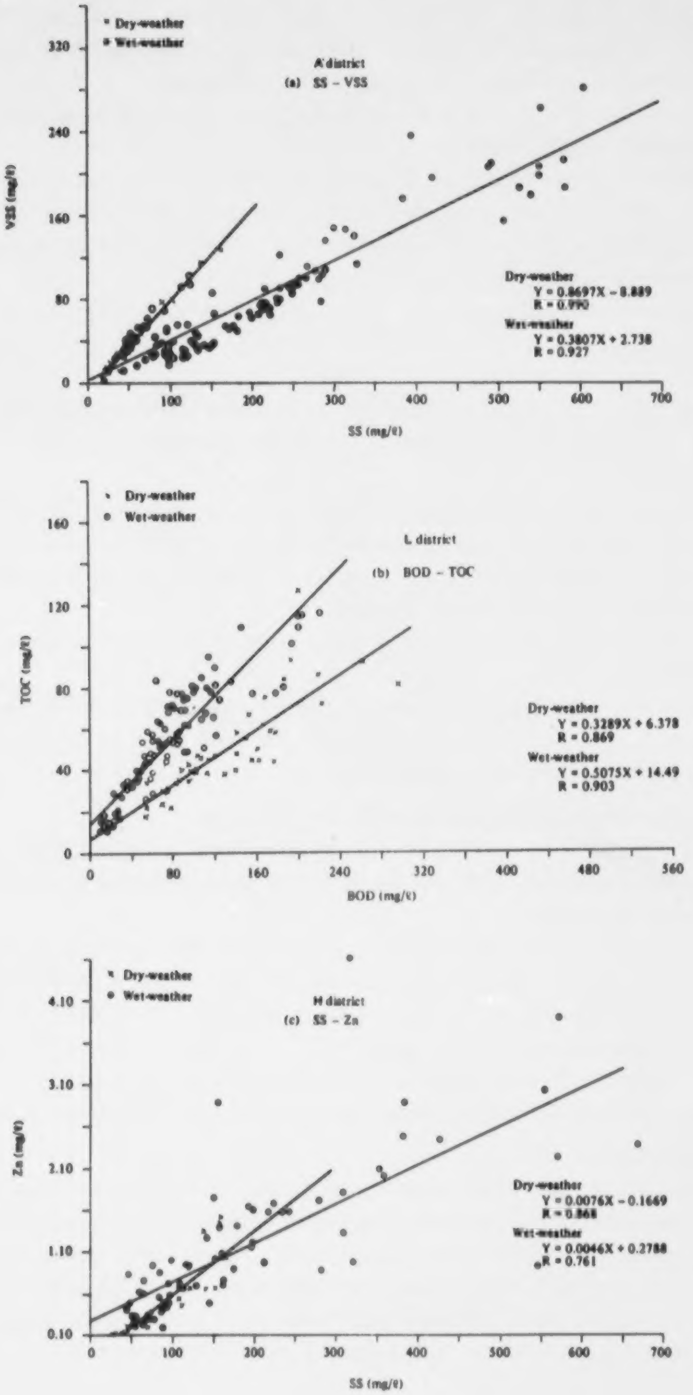


Table 1.6 Summary of Storm Runoff Water Quality from Urban Area

District	Date		Rain-fall (mm)	Wet Weather Average Concentration ($\binom{\text{Upper: Average}}{\text{Lower: Range}}$) mg/ℓ													
				BOD	S-BOD	COD	SS	VSS	T-N	T-P	Zn	Cu	Pb	Cr	Ni	Cd	Coliform Group × 10^4/mℓ
M	1	4.6	8.0	56.7 7.2~142	11.1 5.6~ 17.9	43.9 9.9~ 77.1	420 10.4~ 863	91.9 4~ 226	7.23 3.75~ 18.8	0.24 0.053~ 0.556	0.956 0.11~ 1.75	0.149 0.030~ 0.36	0.277 0.028~ 0.740	0.025 0.004~ 0.046	0.038 0.006~ 0.057	0.0031 0.0014~ 0.0103	0.74 0.037~ 4.0
	2	8.6	19.5	32.1 6.9~ 309	9.24 4.0~ 47.4	43.7 10.5~ 190	303 1.1~ 1180	98.1 9.3~ 460	5.31 1.7~ 25.9	0.24 0.09~ 1.45	0.670 0.06~ 1.80	0.299 0.026~ 1.93	0.212 0.015~ 0.550	–	0.021 0.004~ 0.073	–	0.38 0.081~ 6.2
	3	9.8	11.0	30.5 12.5~ 72	7.69 6.4~ 12.5	40.7 17.5~ 73.5	347 20~ 715	71.2 14.2~ 101	3.45 2.62~ 3.77	0.41 0.36~ 0.89	0.54 0.19~ 0.70	0.12 0.01~ 0.14	0.101 0.018~ 0.130	–	0.018 0.006~ 0.022	–	0.46 0.011~ 0.71
	4	12.4	8.0	62.2 9.7~ 114	14.3 6.1~ 42.3	50.8 10.4~ 80.8	182 16.3~ 280	95.6 8~ 148	8.81 2.28~ 14.9	0.48 0.09~ 1.73	0.734 0.12~ 1.07	0.136 0.031~ 0.192	0.114 0.015~ 0.151	–	0.019 0.008~ 0.027	–	0.88 0.042~ 1.5
O	1	7.19	27.0	1.2~ 12.6	0.9~ 3.5	5.1~ 25.0	38.0~ 490	8.0~ 61.0	1.17~ 4.27	0.01~ 0.24	0.042~ 0.286	0.004~ 0.019	0.010~ 0.055	0.002~ 0.009	0.004~ 0.013	0.0006~ 0.0017	0.057~ 0.76
	2	10.20	9.0	2.5~ 16.1	1.3~ 4.2	7.7~ 25.0	16.3~ 153	7.0~ 35.0	1.18~ 7.46	0.009~ 1.42	0.038~ 0.250	0.0067~ 0.022	0.0099~ 0.034	0.0006 ~0.0018	0.0011~ 0.045	0.0005 ~0.0020	0.22~ 0.76

Fig. 1.8 shows the third rainfall observed in the survey area M. It also shows the relationship between the cumulative loadings of BOD, SS, T-P and K-N in percentage to the respective totals and the cumulative discharge.

The loading hydrograh shows a bell shape, significant of a large loading runoff in the initial stage of rainfall. Just as in Fig. 1.4, however, T-P and K-N are less in such tendency.

Figs. 1.9 (a) and (b) show the changes in water quality in the two survey areas.

While the water quality in the survey area M is very poor, that in the survey area O is very agreeable, proving that the water quality of storm water discharge is largely governed by the local conditions. In such a place as the survey area M where the social activities are vigorous, the effect of storm water discharge on the pollution of waters cannot be neglected, and the separate sewer system may have to be improved some way or other in the future.

Fig. 1.8 Cumulative Flow Volume and Loadings Discharged of Urban Starm Water

M district Sep. 8, 1975

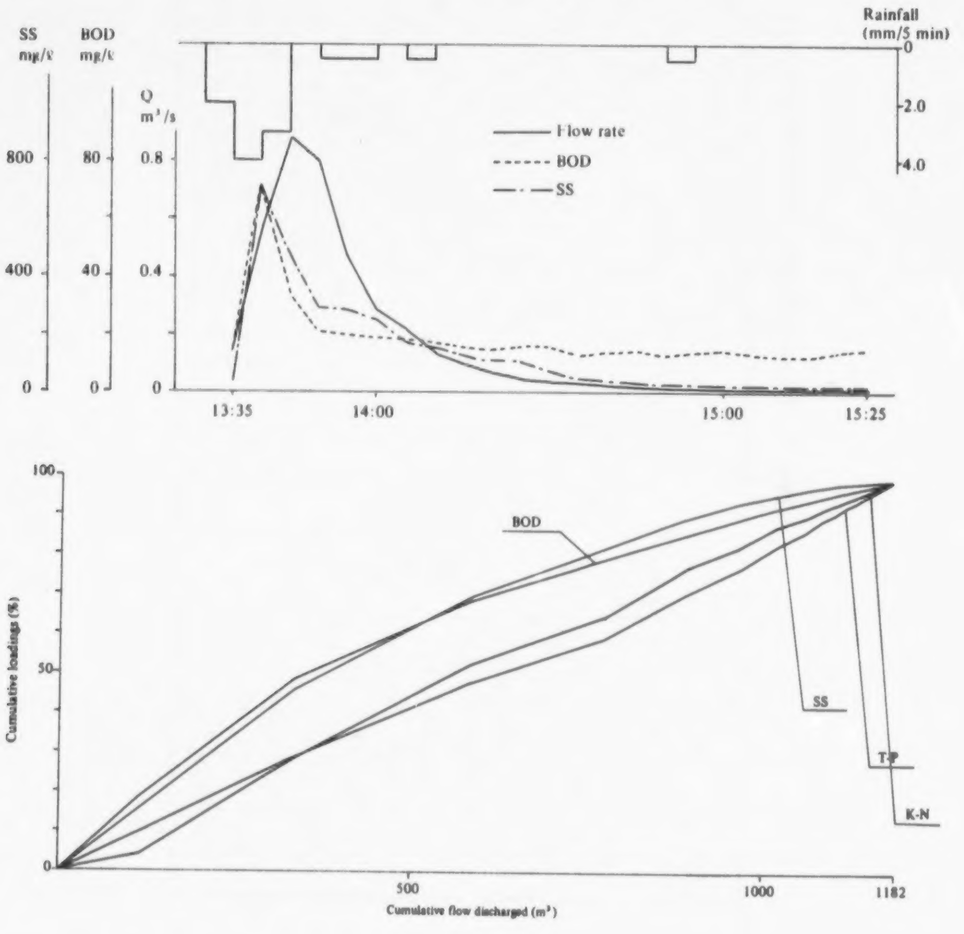


Fig. 1.9 (a) Variation of Water Quality in Storm Water Discharge (Urban Area)

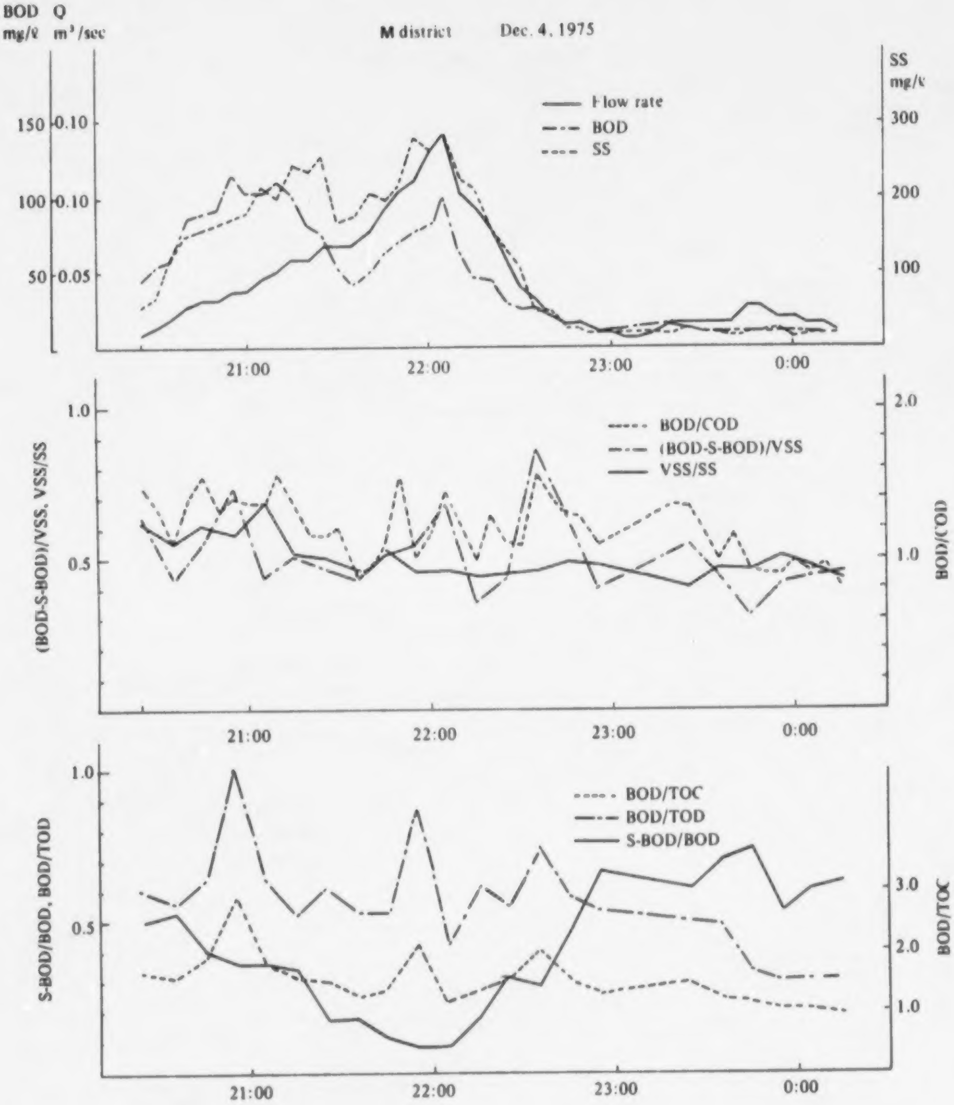
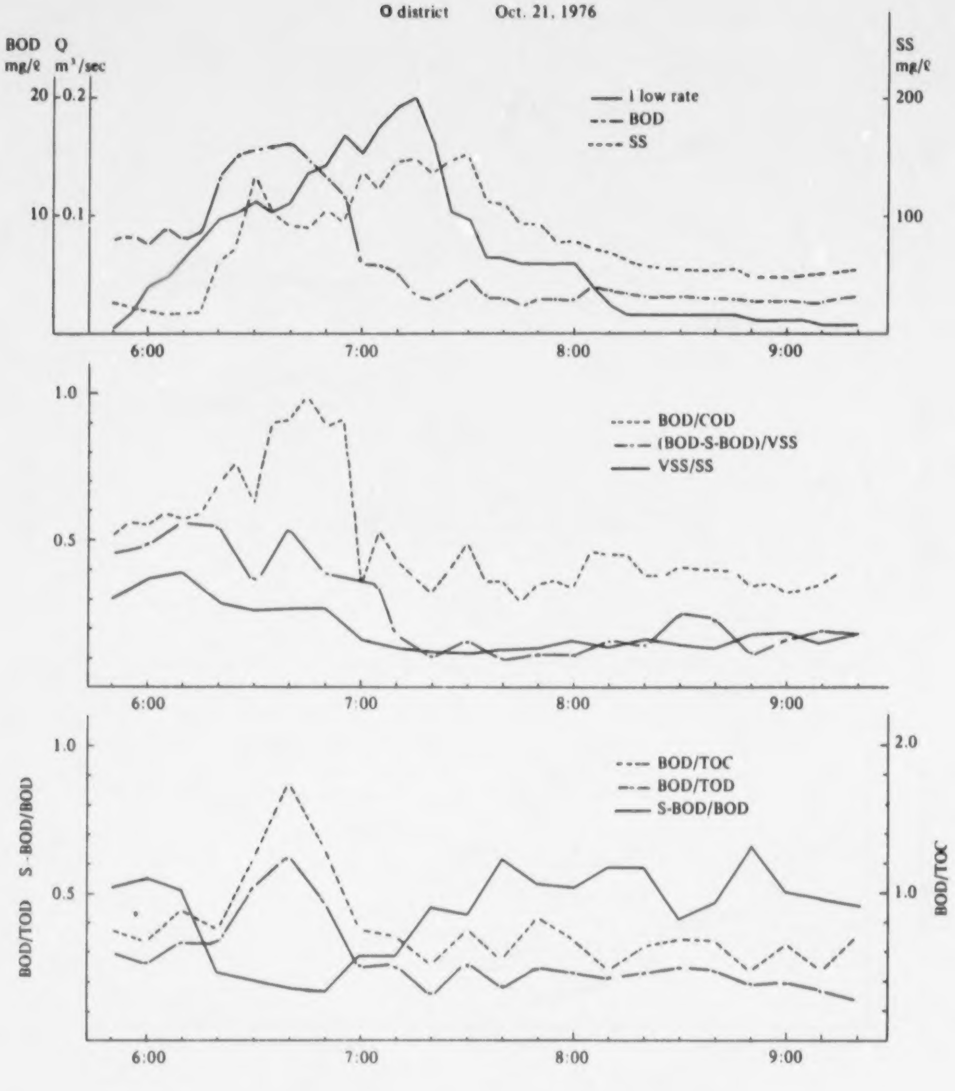


Fig. 1.9 (b) Variation of Water Quality in Storm Water Discharge (Residential Area in the Suburbs)



(b) Characteristics of Water Quality

Fig. 1.10 shows the nitrogen forms in the urban storm water disclosed by the survey of the survey area M.

As the discharge comes close to an end, the ratio of NH_3-N is reduced, while NO_2-N and NO_3-N rise.

Fig. 1.11 shows the correlation between SS and VSS on the one hand and heavy metals on the other.

Interestingly enough, the ratio of VSS/SS is smaller than that in the combined sewer.

There is a significant correlation established between SS and heavy metals. Assuming that all these heavy metals are present in SS, a comparison between them and the heavy metal concentrations usually found in soil shows that Zn, Pb and Cd in SS are more than 50 times those in natural state.

Fig. 1.10 Variation of Nitrogen Forms in Urban Storm Water Discharge

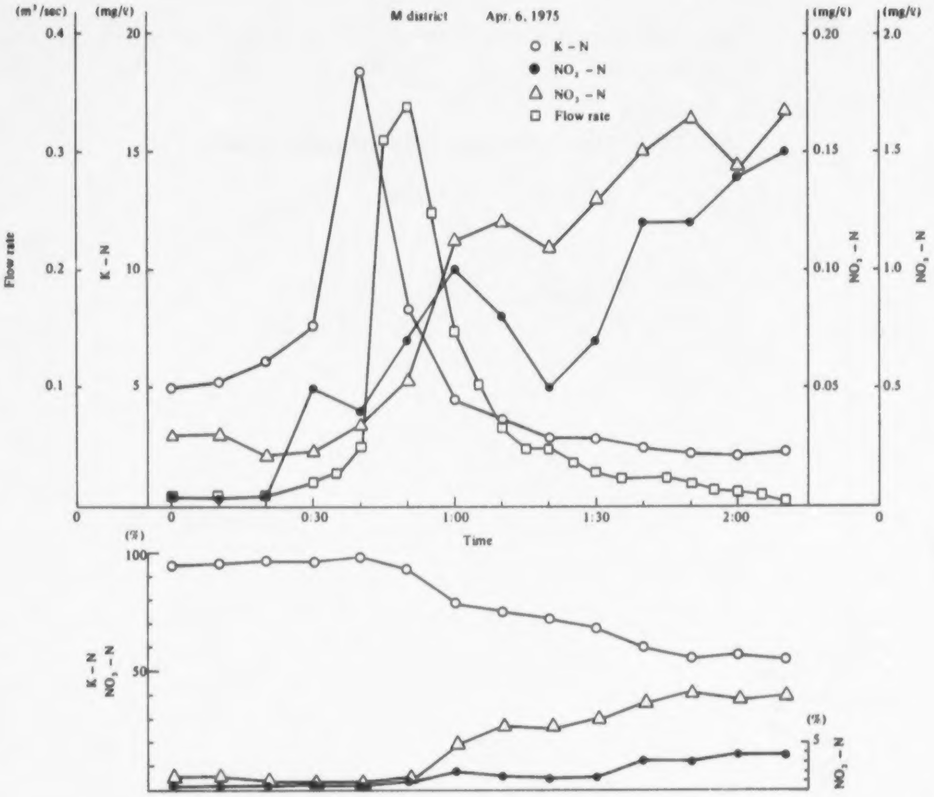


Fig. 1.11 (a) Correlation between SS and VSS

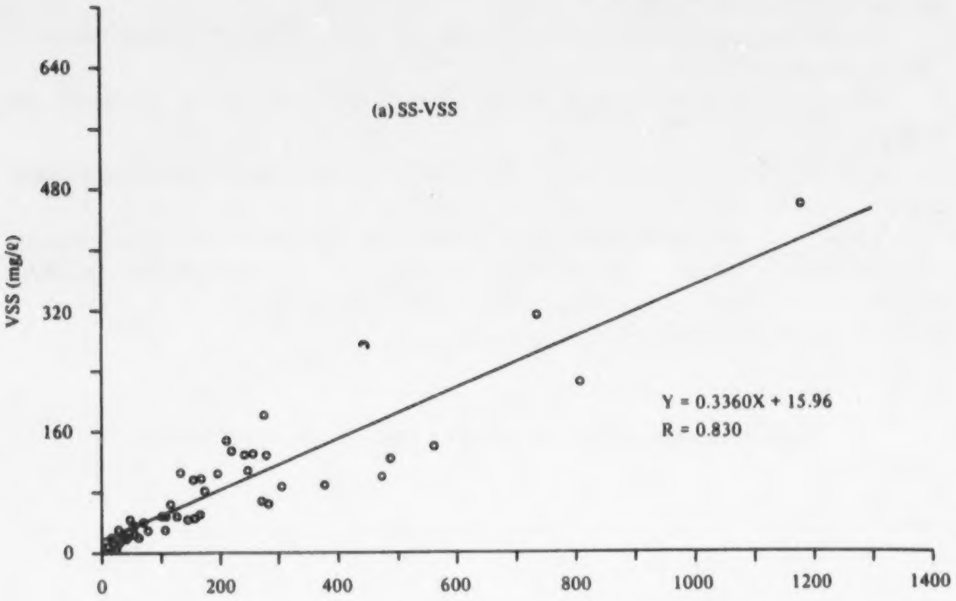


Fig. 1.11 (b) Correlation between SS and Heavy Metals

SS (mg/ℓ)

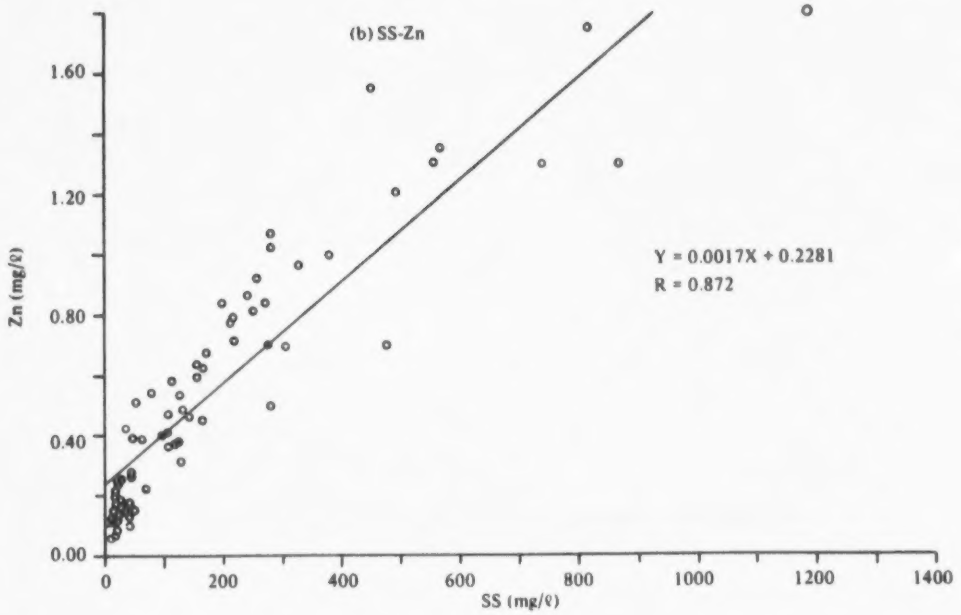


Fig. 1.11 (c) Correlation between SS and Heavy Metals

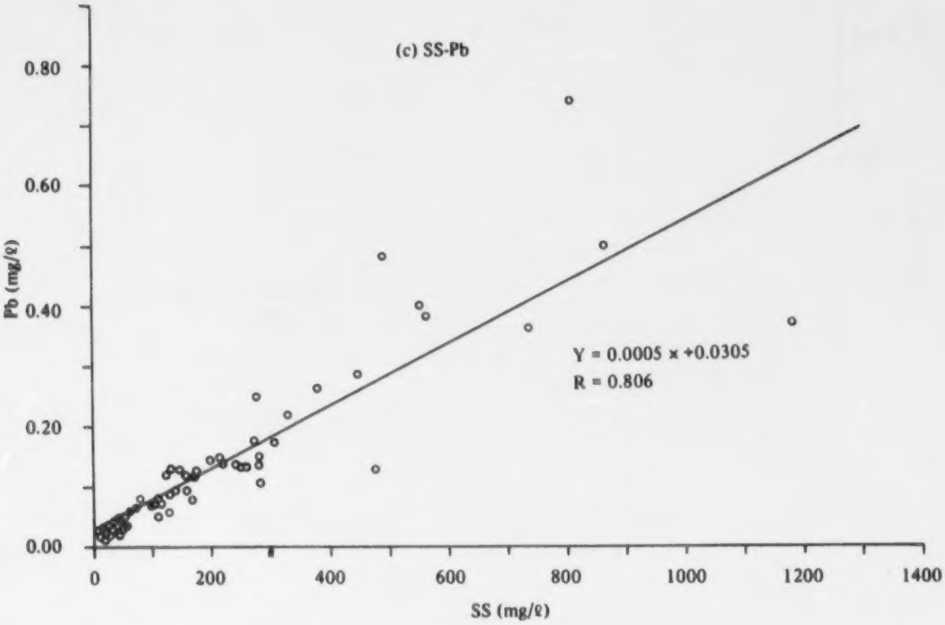


Fig. 1.11 (d) Correlation between SS and Heavy Metals

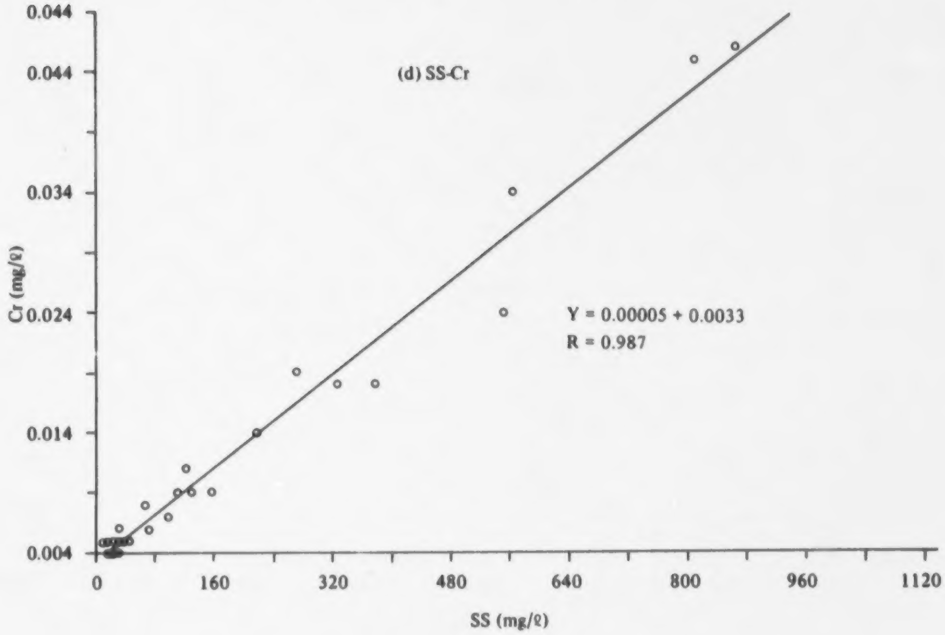


Fig. 1.11 (e) Correlation between SS and Heavy Metals

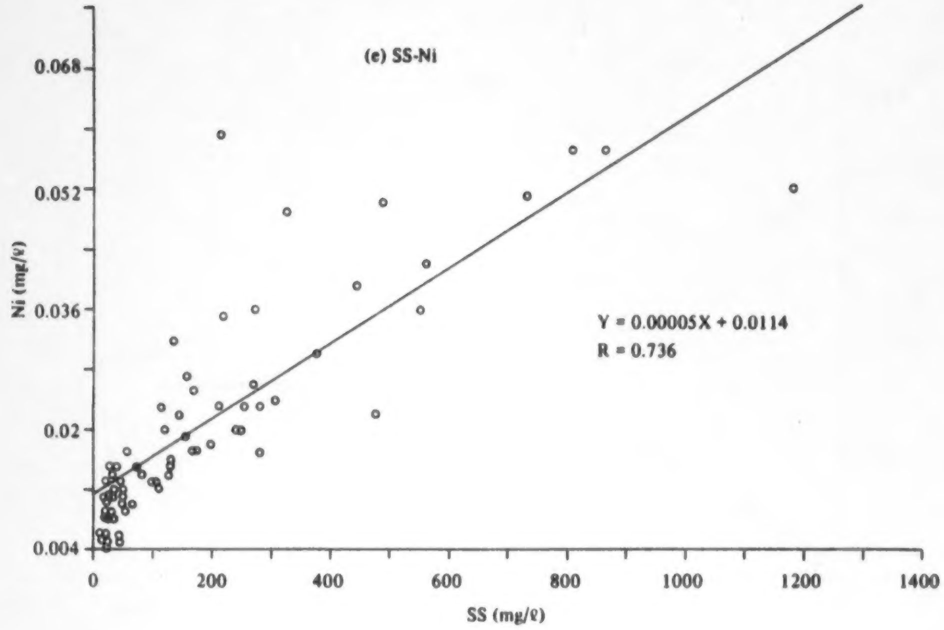
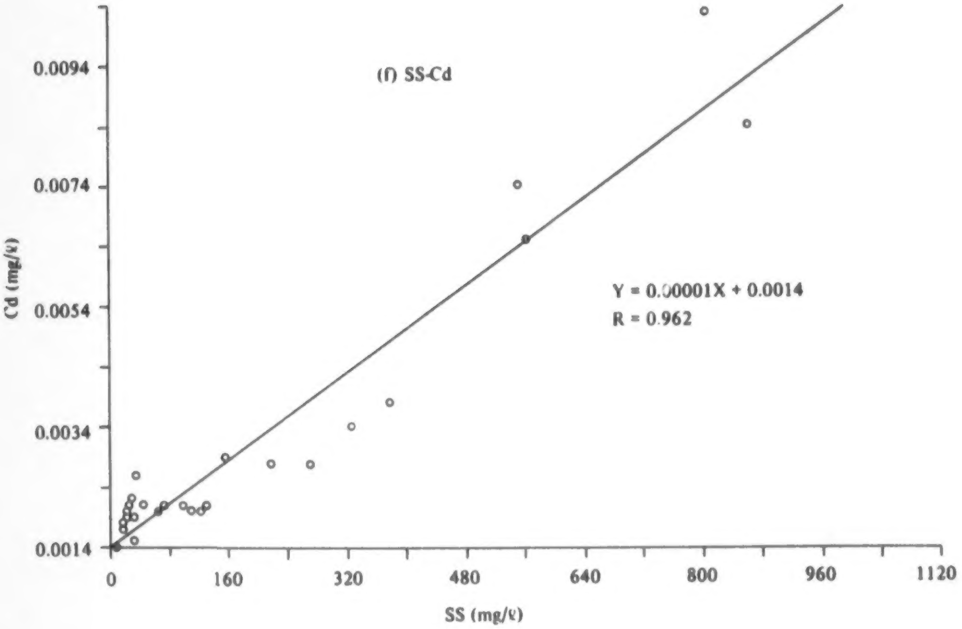


Fig. 1.11 (f) Correlation between SS and Heavy Metals



(c) Characteristics of Rainfall

Table 1.7 shows the results of rainfall analysis made in the survey area M.

In the urban area, the rainfall itself is fouled with air pollutants, including poisonous heavy metals such as Pb and Cd, posing serious threat to human health.

Table 1.7 Water Quality of Rainfall

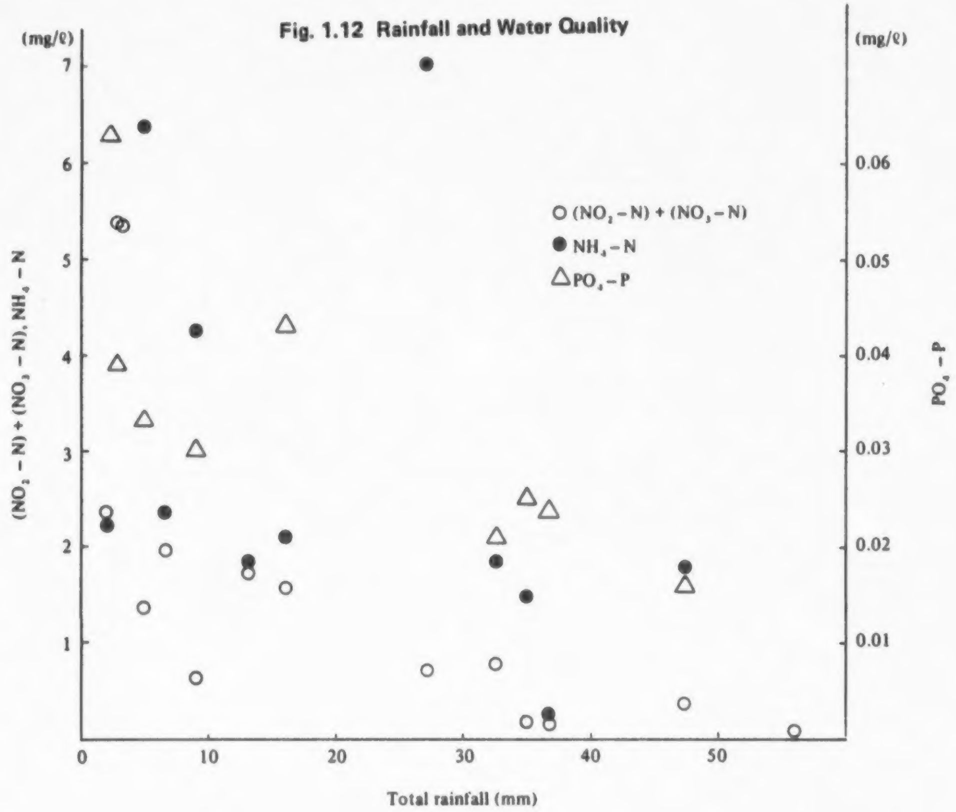
M District April 6, 1975 Rainfall 8 mm

	COD	T-N	K-N	NO_2-N	NO_3-N	T-P	Total residue on evaporation	Volatile residue	Zn	Cu	Pb	Cr	Ni	Cd
Concentration (mg/ℓ)	4.04	0.885	0.53	0.0125	0.343	0.044	79.2	22.2	0.014	0.013	0.0185	0.0026	0.0065	0.0006
Load (g/ha)	323	70.8	42.4	1.0	27.4	3.48	6330	1770	1.14	1.04	1.48	0.208	0.520	0.048

Fig. 1.12 shows the relationship between rainfalls and average concentrations which is obtained by analyzing the quality of rainfall in Tokyo. It is considered that the concentrations of rainfall decline with increase in rainfall to eventually saturate to a constant value.

The results of the surveys in the survey area M and in Tokyo are in agreement so long as the order of values is concerned.

Fig. 1.12 Rainfall and Water Quality



1.3.3.3 Summary of Survey Results

The surveys on combined sewer overflow and urban storm water discharge in the twelve representative cities in Japan have disclosed the following.

1) It is generally seen in wet-weather that BOD, T-P and T-K are diluted significantly while SS remains no nearer being diluted.

2) Wet-weather discharge of T-P and K-N has less to do with the tendency of "first flush" than BOD and SS.

3) In wet-weather, the water quality in the combined sewer gets aggravated seriously, largely different though they may be with locality.

4) In wet-weather the combined sewer experiences an increase in inorganic SS and at the same time an increase in refractory organics.

5) The urban storm water discharge quality varies depending on land use; the discharge from urban areas such as amusement quarters is seriously contaminated.

6) In the urban area, the storm water itself is polluted; the degree of pollution is high, particularly in the early stages of precipitation.

1.4 POSTSCRIPT

A series of surveys of which this is part will be continued till 1973 for collecting more detail data which the authors are confident will provide something of a basis on which to build up more improved sewer systems meeting specific conditions of localities.

CHAPTER 2. COMBINED SEWER OVERFLOW SIMULATION – CASE STUDY ON YABATA SEWER CATCHMENT AREA, TOKYO –

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2. COMBINED SEWER OVERFLOW SIMULATION – CASE STUDY ON YABATA SEWER CATCHMENT AREA, TOKYO –

2.1 INTRODUCTION

A brief mention will be given hereunder to the problem of pollutants overflow from combined sewers. As is well known, the combined sewer transports domestic waste water to the sewage treatment plant during dry weather, and during storm weather, stormwater is added to the domestic waste water. It is impossible, however, to send all the water to the sewage treatment plant because of its limited capacity. Therefore, if storm water flow exceeds a discharge that is two or three times as much as dry weather flow, the excess is all discharged into the river. Originally, it was considered that no harm would result, since the pollutants overflow was diluted to some extent; but upon execution of such a method, it was found that more polluted water than dry weather flow owing to released deposits on the sewer was discharged into receiving waters. Regarding the magnitude of the problem, it is said in the United States that the released pollutants load by storm water into receiving water should be more or less equal to those from the secondary treatment plant (in terms of BOD). Because of this, the Japanese government is now directing cities to install separate sewers. However, since sewers in existing major cities like Tokyo, Nagoya and Osaka are of the combined sewer type, this problem has recently been taken up to be solved. Nevertheless simulation of pollutants overflow was scarcely attemped in the past, because the pollutant runoff pattern itself occurred not in a simple manner; and so far no adequate measures have been taken up.

The Urban River Section of River Division, Public Works Research Institute, has been carrying out the investigation of storm water runoff from the urban area since its inception 1969 with the cooperation of the Tokyo Metropolitan Government. From 1972 onward, the section has also been sampling and analyzing combined sewer overflow. Fortunately, the section was able to propose the simulation model of storm water runoff in 1973 and succeeded in the simulation of the pollutants runoff in 1975. Although the betterness of fit of the simulation still requires to be ascertained by applying it to the other basin, it has been decided to report on the simulation as it is at the present stage, to solicit the criticism of readers.

It may be necessary to be added here that the present report is concerned only with BOD.

2.2 OVERFLOW QUALITY CHARACTERISTICS

Since there are many detailed reports on the characteristics of overflow from the combined sewer, only their essence is extracted here for lack of space.

First, Fig. 2.1 ~ 2.3* show examples of observed hydrograph, pollutograph, and concentration of BOD at the Yabatagawa Basin. In these figures, it may be pointed out that conversion from rainfall to runoff (Q: m^3/s) indicates comparatively simple correspondence, but then the relation between the concentration of BOD (C: mg/l) and BOD load discharge (Qs = CQ: g/sec) is somewhat complicated. As a result, sampling intervals, from the initiation of runoff to the peak is short. For comparison, patterns during dry weather are also indicated by broken lines.

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* The peak discharge of 49 m^3/sec. on 10th November, 1973 in Fig. 2.2 is nearly equal to the design discharge.

Fig. 2.1 Example of Observed Overflow Characteristics (Yabatagawa, Tokyo)

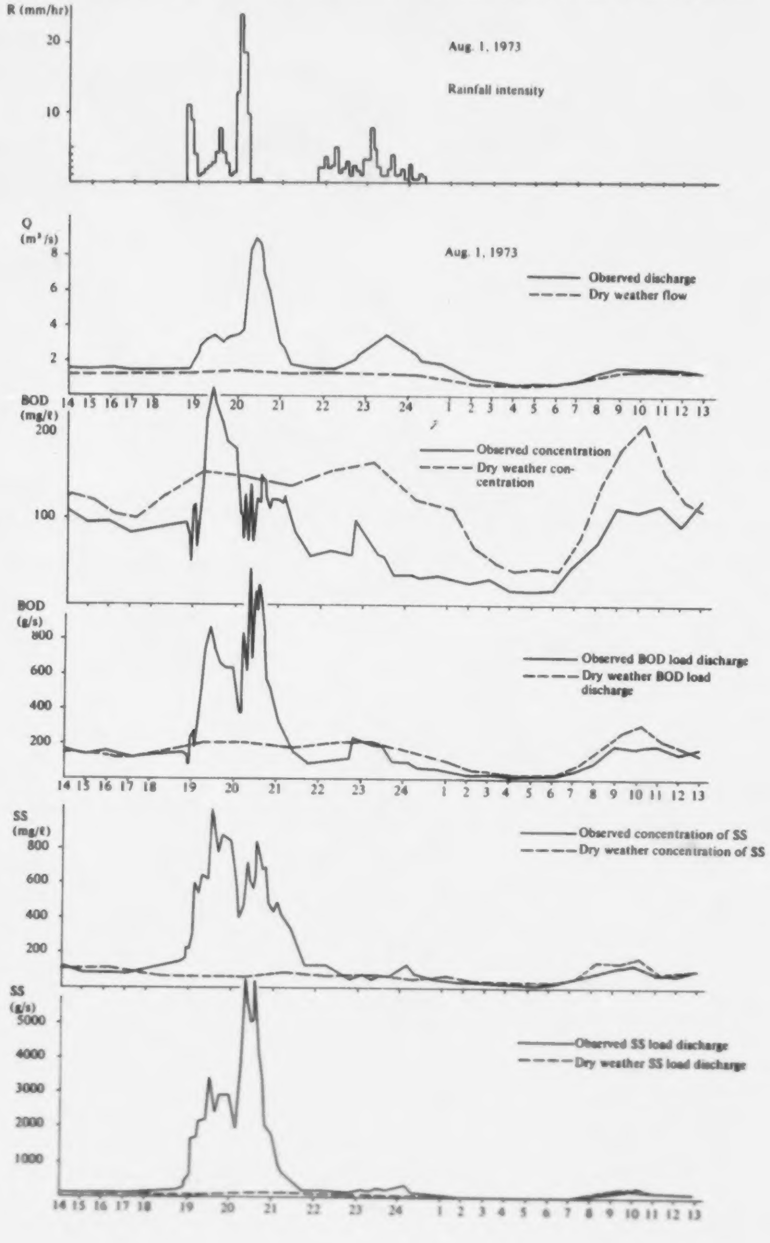


Fig. 2.2 Example of Observed Overflow Characteristics (Yabatagawa, Tokyo)

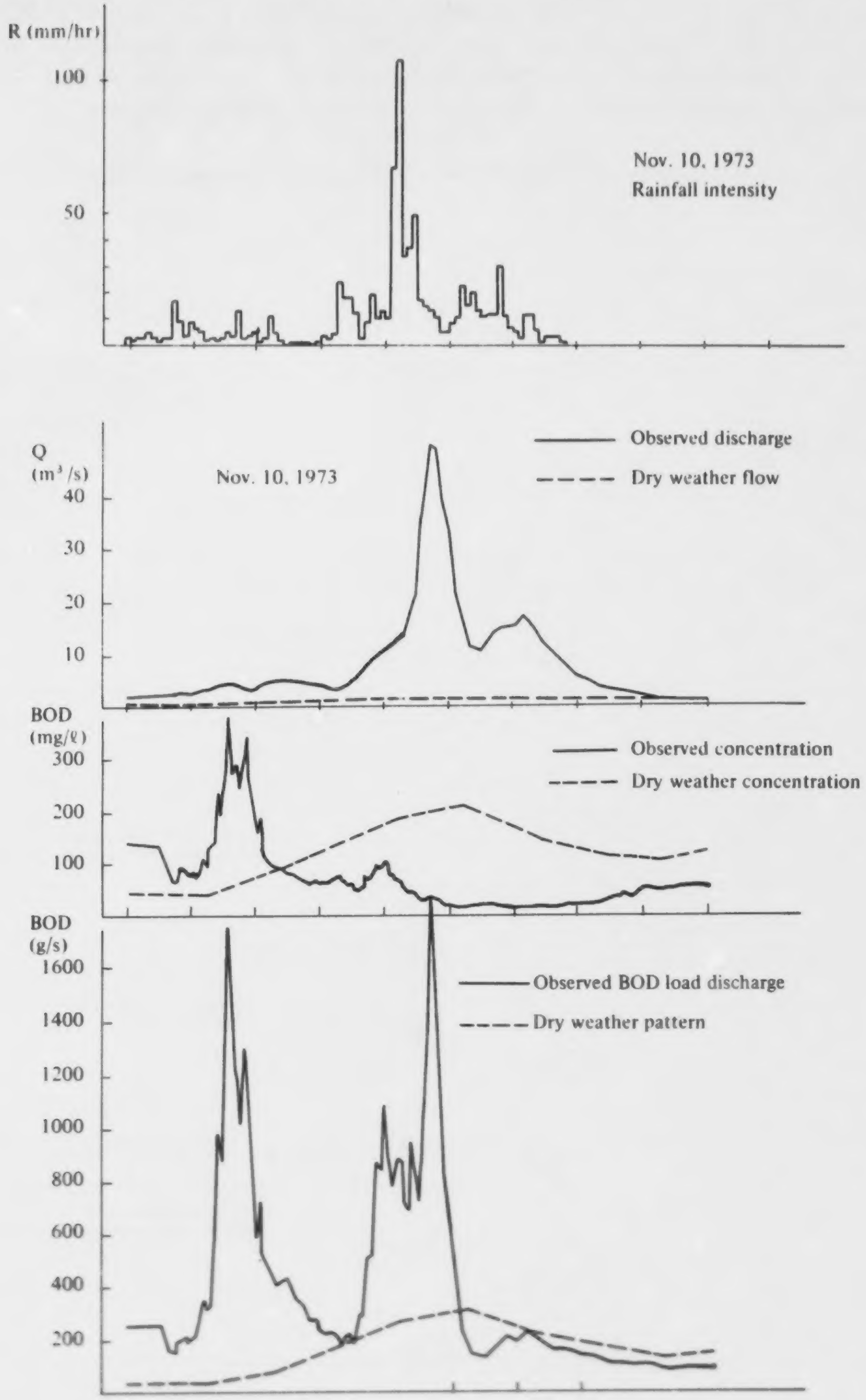
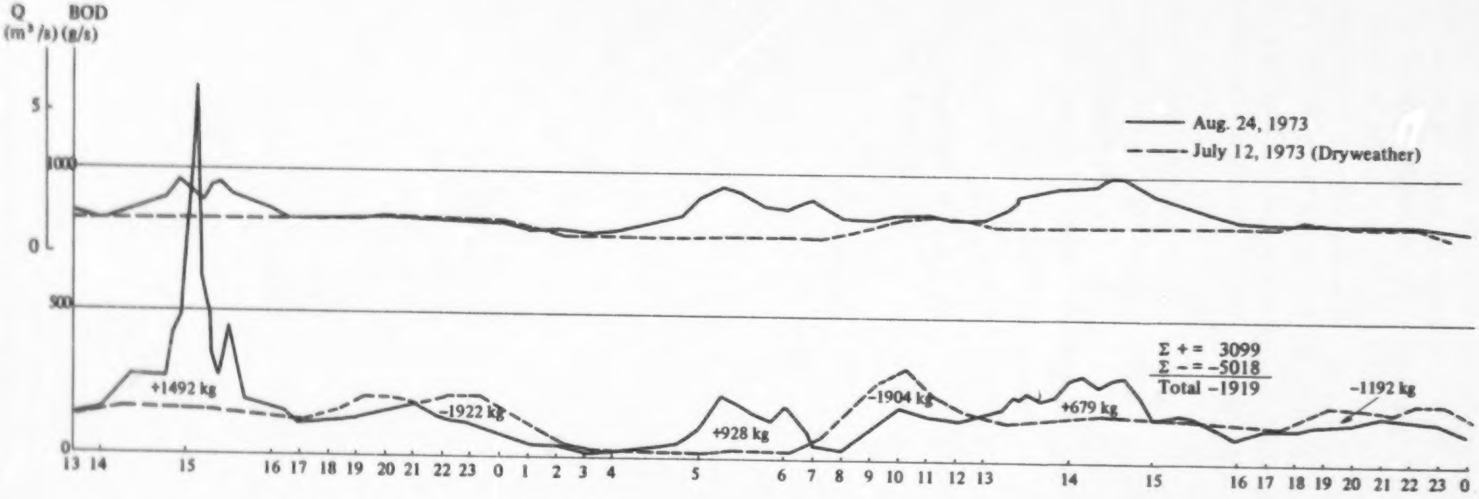


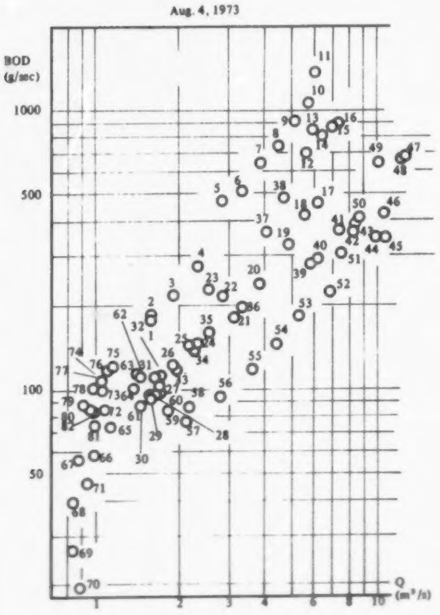
Fig. 2.3 Example of Observed Overflow Characteristics (Yabatagawa, Tokyo)



These exmples clearly show the high BOD concentration at the initial period (so-called "first flush") and the condition in which BOD load discharge does not decrease even if the magnitude of runoff increase (this condition cannot be explained by dilution alone). As for the reason, a concept is generally accepted that deposits accumulated on the sewer during dry weather are transported by storm water. In this respect, the Report on Storm Water Investigation at Northampton* is famous for verifying this concept by discharging a large quantity of clean water into the sewer during dry weather. Pollutants from the ground surface will be dealt with in Chapter 2.3.

The observed values of BOD load discharge are plotted with Q on a loglog scale according to the sequence of observations, and the curve naturally indicates a clockwise loop (Fig. 2.4 ~ 2.6). This phenomenon is understood to be the result of a decrease in the pollutant load deposited in the sewer (S), by storm water.

Fig. 2.4 BOD Load Discharge ~ Q
(Yabatagawa, Tokyo)


Aug. 4, 1973

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* This report is one of the classics on this problem and worthy of a perusal.

Fig. 2.5 BOD ~ Q (Yabatagawa, Tokyo)

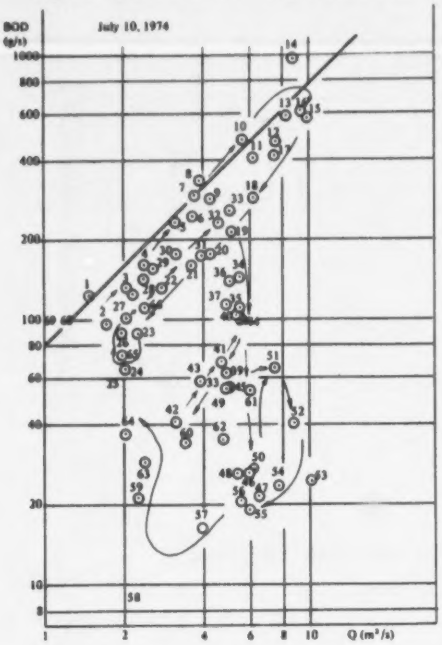
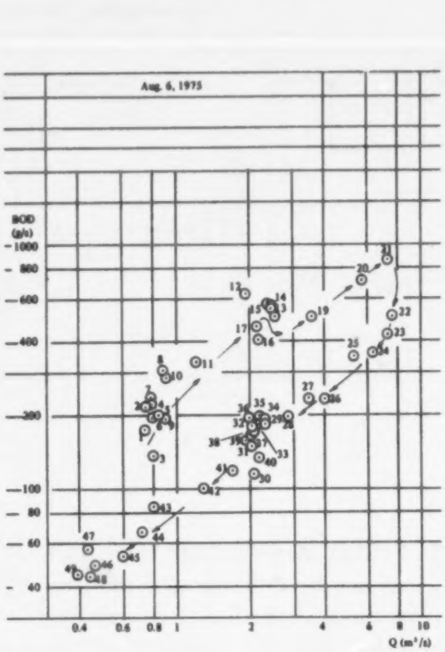


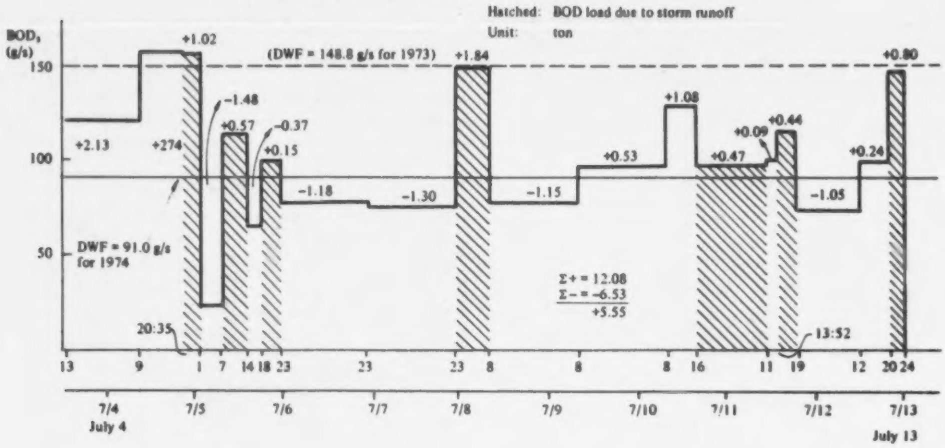
Fig. 2.6 BOD ~ Q (A city)



Another important phenomenon is that the pollutant load discharge during about one or two days after the termination of runoff becomes smaller than the dry weather load discharge. Considering that this phenomenon was found in older records, the author sampled and analyzed the load discharge for 12 to 24 hours after the termination of runoff in fiscal 1973 and this phenomenon was observed in every example analyzed. For the interpretation of this phenomenon, variation in the dry weather load can be considered, but according to the result of observations so far conducted, the dry weather pattern is comparatively stable, if a season is fixed. Therefore, the cause of the phenomenon is considered to be re-accumulation of dry weather load, that is, the recovery of the deposits. This accumulation may occur in the following way: Judging from the fact that the accumulation also seems to occur during later stage of runoff, it is inferred that gravel mounds and pits located on the sewer pipe act as a sort of load accumulation potentials, and even if the discharge at the later stage of runoff is a little greater than dry weather flow, pollutant load may accumulate on the gravel mound and in the empty pits on the sewer pipes. This can be easily understood if you imagine gravel pits of catch basins.

Finally, as for the pollutant load balance for one rainfall or during a certain period, 10 days' continuous water sampling and analysis were carried out for fiscal 1974 and 1975 on the basis of the survey results for fiscal 1973. The result of the field survey for fiscal 1974 is shown in Fig. 2.7. Regarding BOD, the pollutant load is more or less balanced, if +4.87 tons for about 1 day from 3rd to 4th July, 1974 is ignored. The same results were obtained in the survey for fiscal 1975. From the above, it is observed that pretty high accuracy can be expected of the simulation of BOD, even if supply of pollutants from the ground surface is ignored.

Fig. 2.7 Comparison with Dry Weather by 10 Days Observation (Yabatagawa, Tokyo)



2.3 FIELD SURVEY OF THE POLLUTANTS ON THE SOURCE

As can be understood from the above-mentioned observation, deposits of pollutants on the basin and, particularly on the sewer, are playing a considerably important role. For this reason, a field survey of pollutants on the sources has been carried out. Yabatagawa basin was also chosen for the field survey. The area is 5.4 km² and is mainly residential. Its population is about 140,000 and population density is 260 person/ha. Detailed explanations on the survey are omitted here, and only the results of the survey so far conducted are shown in Table 2.1. An example of the survey is shown below for reference. In the case of survey on sewer manholes, 300 manholes were randomly sampled out of 6,000 manholes in the basin concerned and the number of the existence of pollutant deposits was checked and the quantity and quality of part of deposits were measured, thereby estimating the pollutant load for 6,000 manholes. In the case of the street surface, pollutant load was estimated from water sprinkling tests at only three places. From the results of the survey, it was found that the load potential at this point of time was around 5 tons and the deposits on the sewer accounted for the main part of the load potential, as shown in Table 2.1.

Table 2.1 Sources of BOD Load in the Basin (Surveyed)

Location	Accumulated Load (ton)
Street Surface	0.66
Street Inlet	0.77
Sewer	0.33
Sewer Man Holes	2.5
House Inlet	0.0
Pervious Area	0.05
Total	4.31

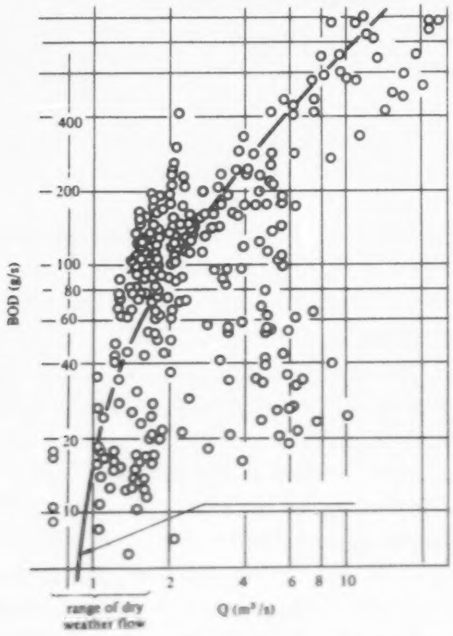
2.4 SIMULATION OF BOD LOAD DISCHARGE

Various characteristics which were used for simulation are briefly described here in the order of the equation of motion in items 2.4.1 and 2.4.2 and the equation of continuity for pollutants in item 2.4.3.

2.4.1 RELATIONSHIP BETWEEN DISCHARGE Q AND POLLUTANT LOAD DISCHARGE Qs

It was pointed out by the author earlier that the relation between discharge Q and pollutant load discharge Qs can be expressed by $Qs \propto Q$ for BOD if the loop is ignored and the result is averaged, and $Qs \propto Q^2$ * for SS, in which the loop characteristics are weak owing to its properties. In dry weather, however, these relationships were not strictly maintained and particularly at night, a rapid decrease of Qs became remarkable. All the observed values for fiscal 1974 totaling 500 are plotted in Fig. 2.8. Judging from this figure, it is clear that something like a concept of critical tractive force in sediment load had better be introduced.

Fig. 2.8 BOD ~ Q (for 1974)
(Yabatagawa, Tokyo)



The relationship can be expressed by the following equation.

$$Qs = K'(Q - Qc) \quad \cdots\cdots\cdots\cdots \quad (1)$$

It is better not to think too seriously about what discharge should be taken as Qc. Just consider as Qc the lowest hourly discharge at the location concerned at dawn when the flow rate at each point in the basin drops and pollutants become difficult to move.

---

* This expression agreed with the formula for suspended sediment discharge. For instance, refer to "Applied Hydraulics" Vol. 2.I, p.27.

2.4.2 INTRODUCTION OF BASIN RESIDUE LOAD (S)

As pointed out earlier in the item dealing with overflow quality characteristics and taking into consideration the results of water sprinkling tests on the street surface and sewer conducted by our section, a concept of basin residue load (S) is introduced and Qs can be expressed as follows;

$$Qs = K'' S^m \quad \text{.......} \quad (2)$$

The quantity S can be defined as the total load in the basin that can be discharged out of the basin at each instant, and as can be seen from eq. (2), the smaller S becomes, the smaller will be Qs. Particularly when m > 1, the decrease becomes very rapid. Values K'' and m will have to be obtained by simulation. Naturally this value S should not widely differ from the results of the field survey of pollutant sources mentioned earlier. From items 2.4.1 and 2.4.2, we can obtain an equation of motion as shown below:

$$Qs = K S^m (Q - Qc) \quad \text{.......} \quad (3)$$

2.4.3 EQUATION OF CONTINUITY

Next, we need an equation of continuity, i.e., an equation which traces variation in time of the basin residue load.

This equation can be written as follows:

$$dS/dt = DWF - Qs \quad \text{.......} \quad (4)$$

and by approximation

$$\Delta S = S_{t+\Delta t} - S_t = (DWF - Qs)\Delta t \quad \text{.......} \quad (4')$$

$$= \left(\frac{DWFt + \Delta t + DWFt}{2} - \frac{Q_{s,t+\Delta t} - Q_{s,t}}{2}\right)\Delta t \quad \text{.......} \quad (4'')$$

In eqs. (4) ~ (4''), DWF means dry weather BOD load discharge into the basin (Input), and observed values of BOD load discharge in dry weather are substituted for this supply because there is no other proper data of this kind.

2.4.4 INITIAL CONDITIONS (S_0)

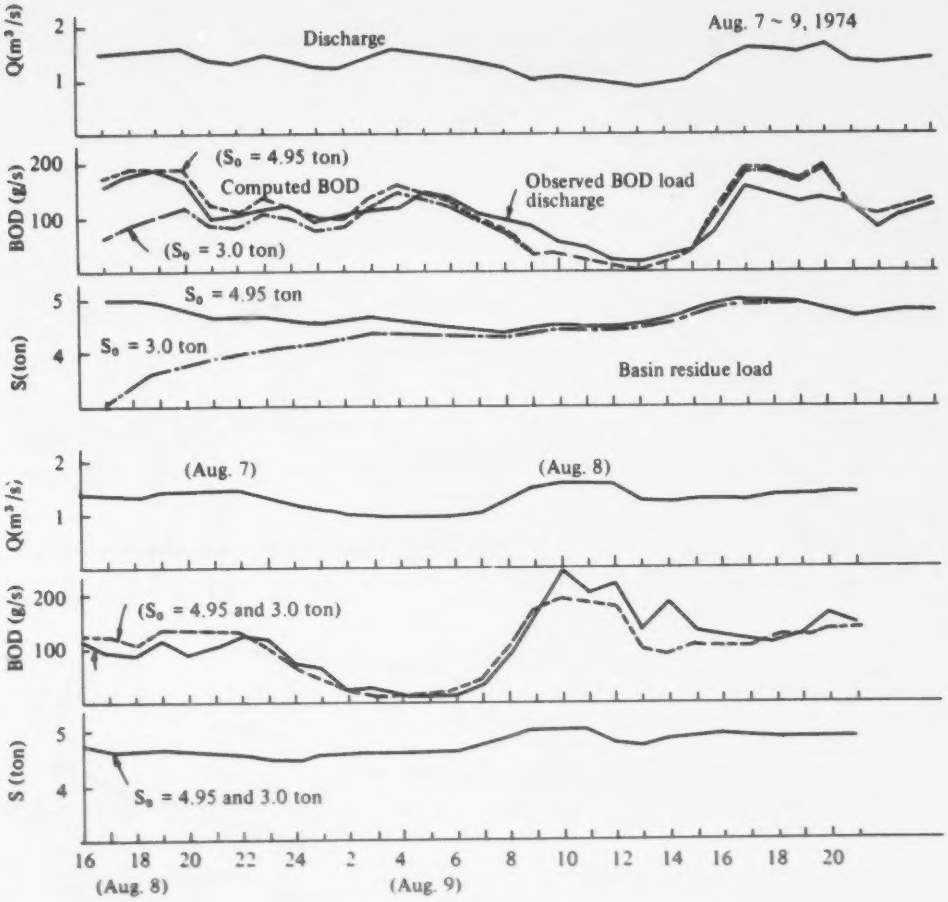
Eqs. (3) and (4) are already given. If Q is given from the observed value or rainfall runoff analysis, and an initial basin residue load (S_0) at the starting point of calculation is given, calculations can be made successively.

The value S_0 was obtained this time in the following way:

Namely, first, the values of exponent m and K with respect to pollutograph in storm weather were obtained approximately, and these values were applied to the data in dry weather (16 to 18 July, 1974). Then it was found that an equilibrium was reached within about 1 day irrespective of the magnitude of S_0 at initial time, and an S-curve in dry weather was obtained. The values of constants were obtained by a trial and error method and first exponent m was almost uniquely determined from the load discharge characteristics in storm weather and then there was almost no room for changing K. This was probably due to so many constraints such as average load and amplitude in dry weather.

The result of simulation is shown in Fig. 2.9. In Fig. 2.9 the range of S spreads from 4.5 tons to 5 tons. This figure unexpectedly shows near correspondence to the result of the field survey of pollutant sources.

Fig. 2.9 Simulation of Dry Weather BOD Load Discharge (Aug. 7 ~ 9, 1974) (Yabatagawa, Tokyo)



If values of K and m are determined, there is another way to determine the value of S_0 which will agree with the observed value at the initial stage (sometimes this method gave better agreement).

2.4.5 RESULTS OF SIMULATION

Figs. 2.10 ~ 2.14 show the results of simulation with m = 2.0, K = 11.43, Qc = 0.87 m³/s (for fiscal 1974) and m = 2.0, K = 4.12, Qc = 0.6 m³/s (for fiscal 1973) and using S at the same clock time in dry weather of Fig. 2.9 for S_0. Fig. 2.10 shows the results of continuous sampling for 10 days from 3rd to 12th July, 1974 and gives comparatively satisfactory results. It indicates good agreement for storm weather and clearly expresses the decrease in BOD load discharge (recovery of deposition) after rainfall. Values of observed discharge are omitted from Fig. 2.10 for lock of space. Variation of S in time is also shown in Figs. 2.11 ~ 14, and rapid decrease during storm and slow recovery can be seen.

Fig. 2.10 Simulation Result of 10 Days' Observation (Yabatagawa, Tokyo)

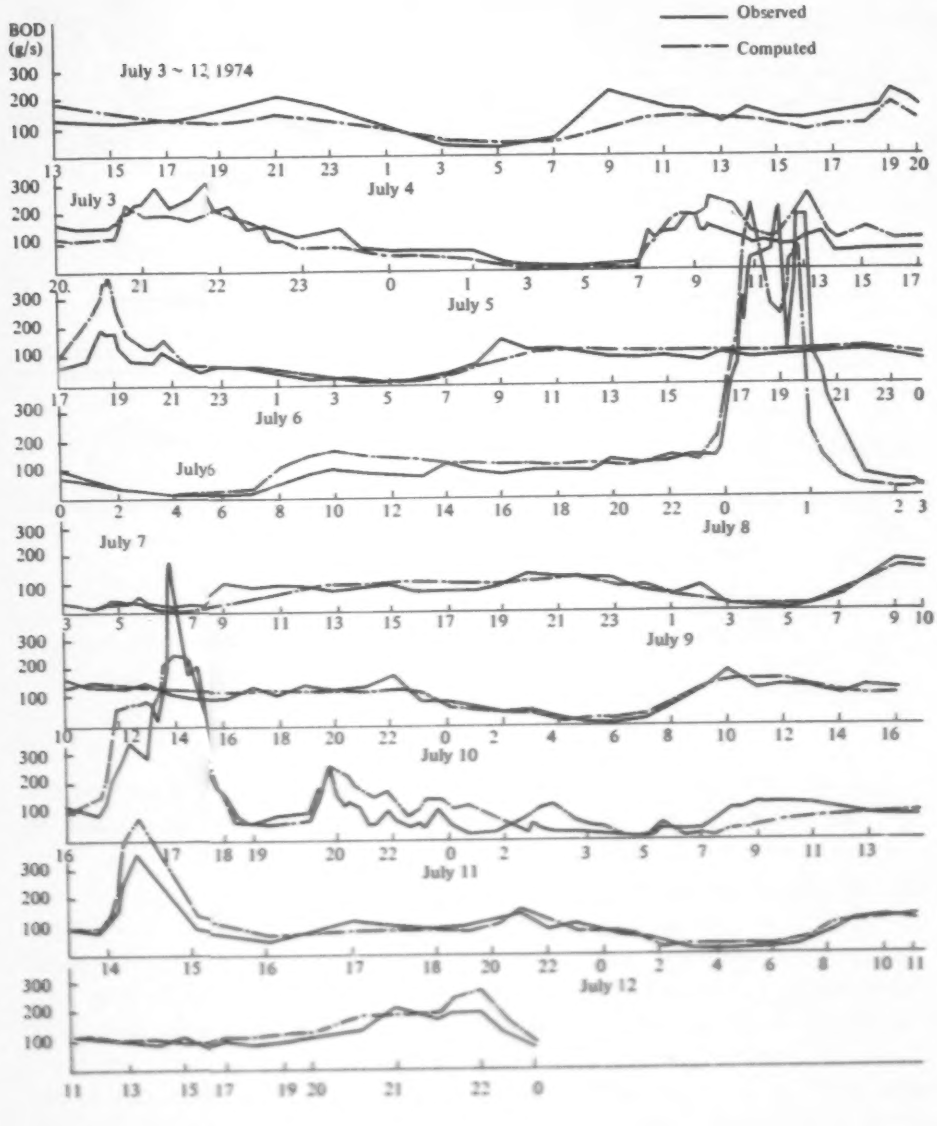


Fig. 2.11 Simulation Result (Yabatagawa, Tokyo)

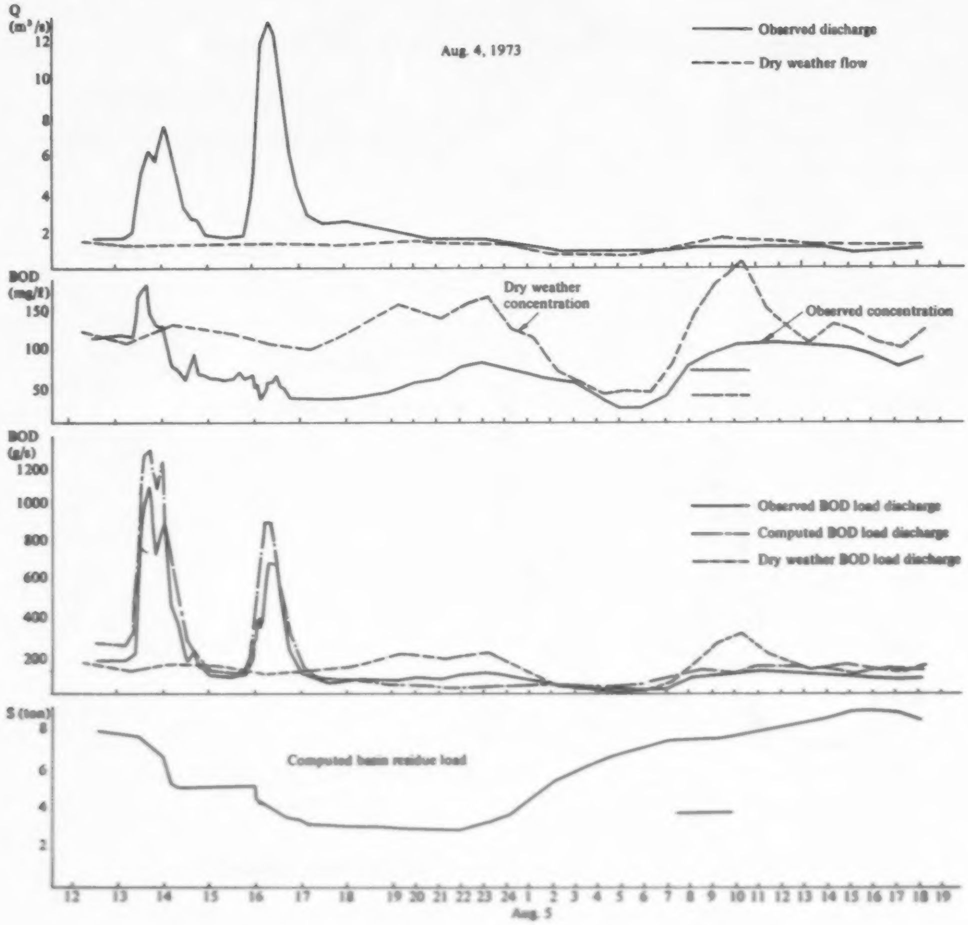


Fig. 2.12 Simulation Result (Yabatagawa, Tokyo)

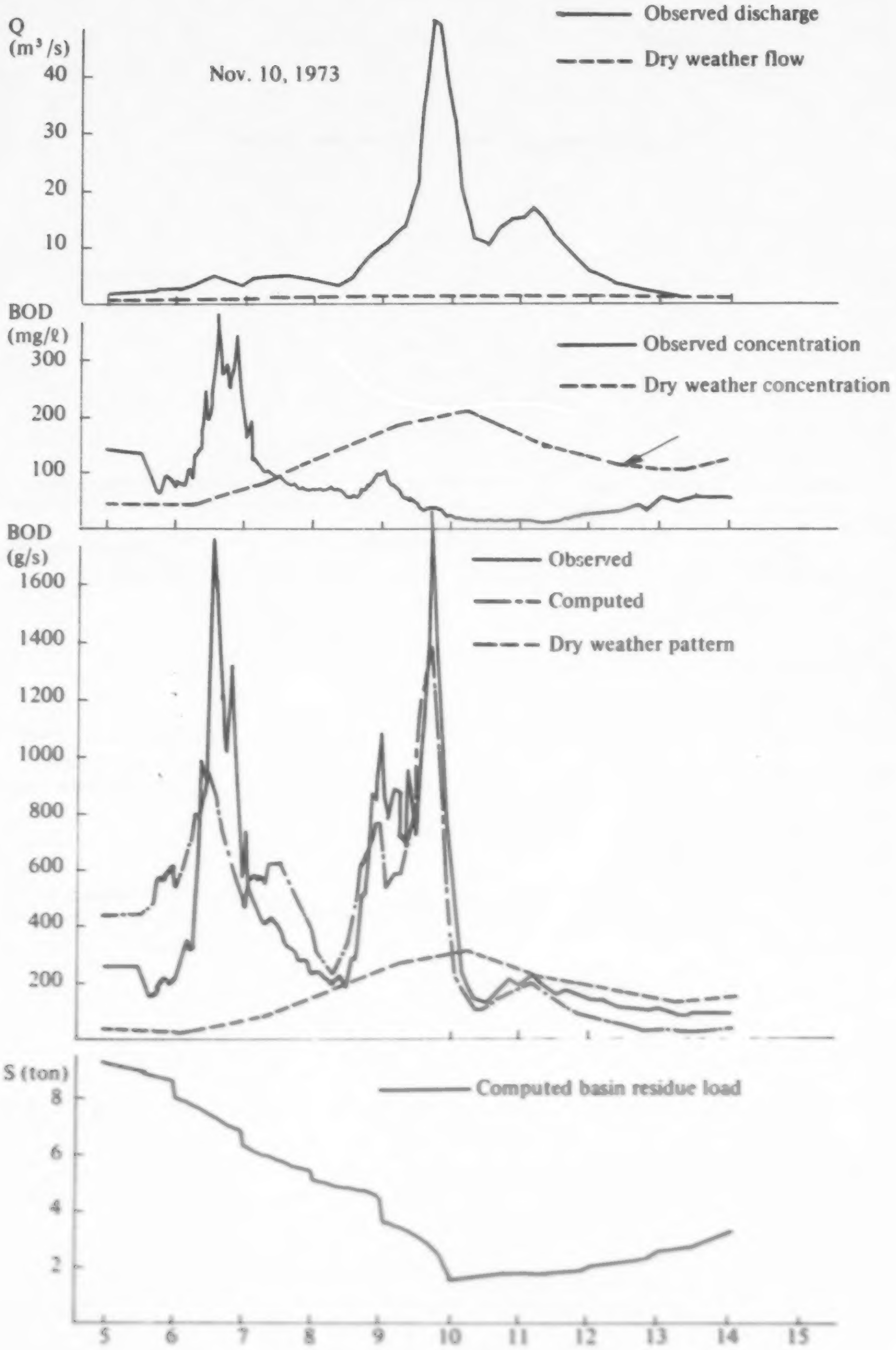


Fig. 2.13 Simulation Result (Yabatagawa, Tokyo)

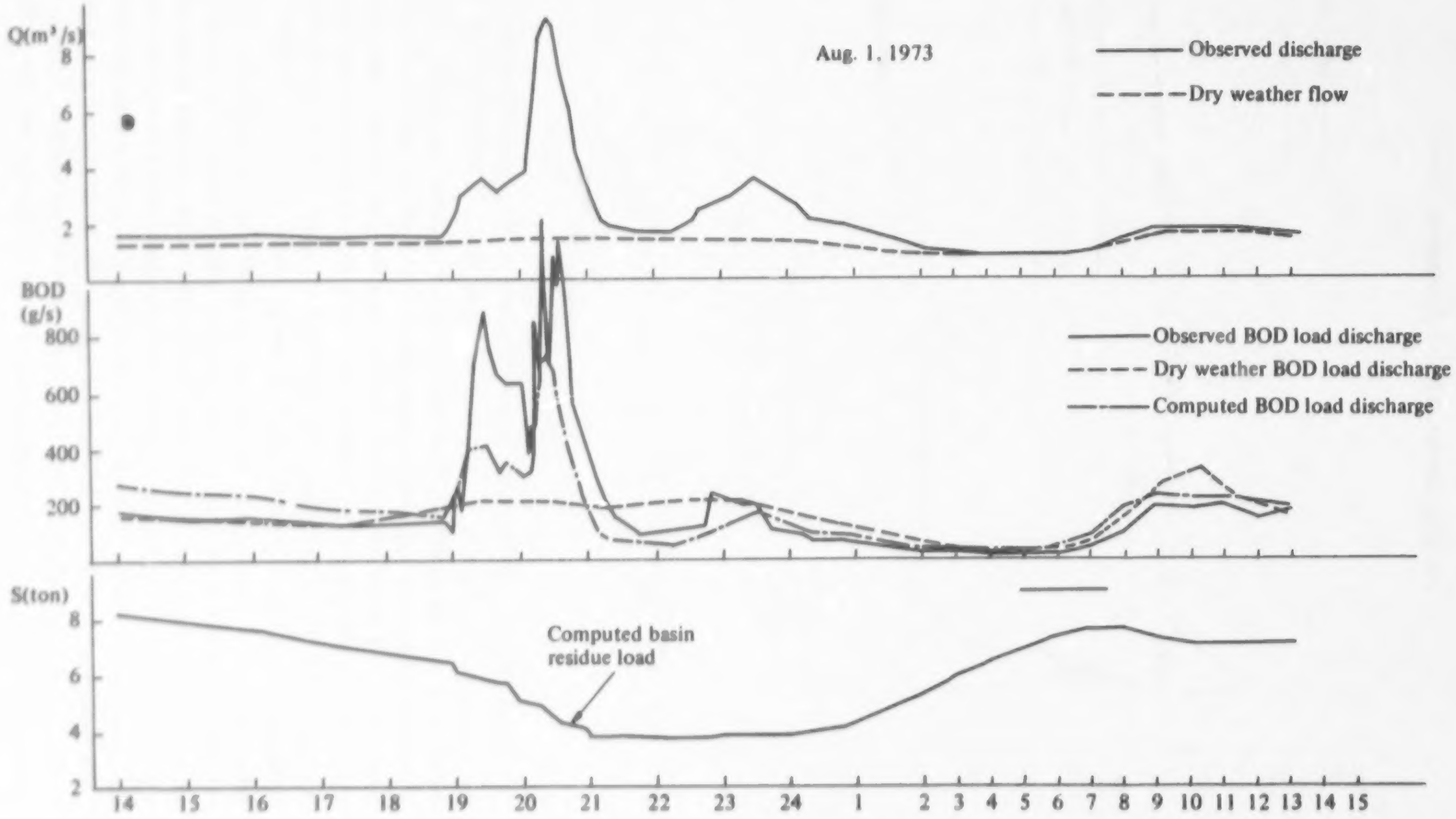
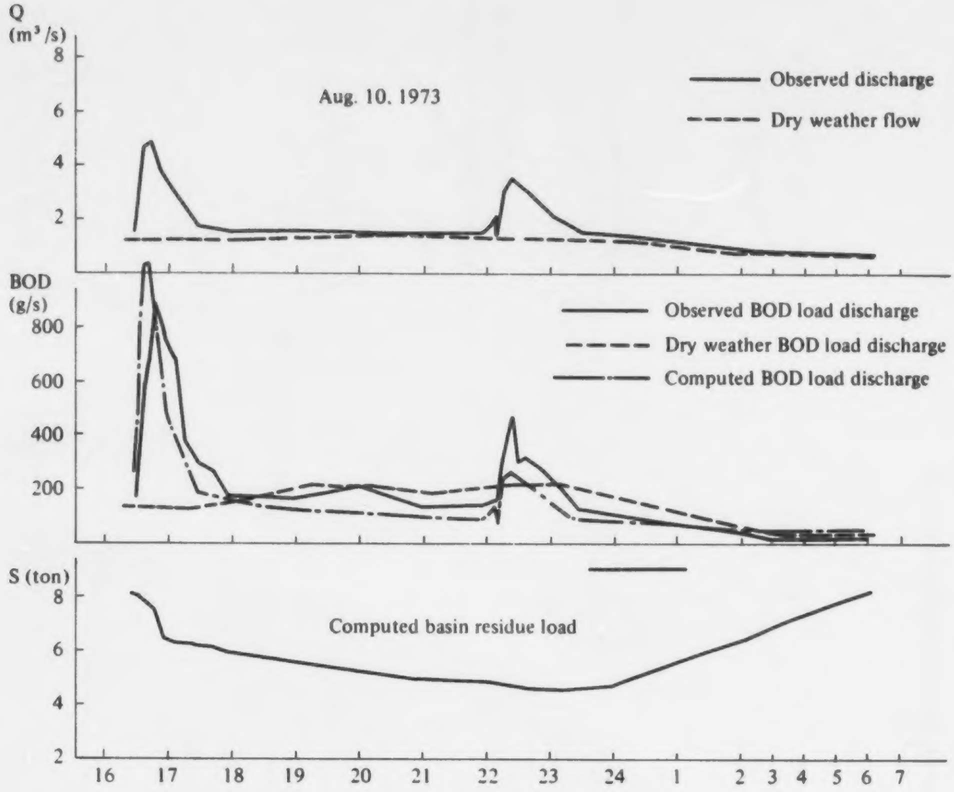


Fig. 2.14 Simulation Result (Yabatagawa, Tokyo)



What poses a problem here is the difference in the values of constants between the simulation for fiscal 1973 and that for fiscal 1974. A little examination on this matter here will be justified. What should be first pointed out is that there is a difference between load discharge characteristics both in dry and storm weather for fiscal 1973 and fiscal 1974. First for dry weather, the average load discharge in fiscal 1973 is 149 g/s (BOD), whereas that in fiscal 1974 is 94 g/s (see Fig. 2.7), and the reason for this is unknown. The petroleum crisis may be a partial cause, but is considered insufficient to cause such a change. In addition, dry weather flow has hardly changed. The method of chemical analysis does not offer any cause. Next for storm weather, total load for one rainfall event for fiscal 1973, for instance, is 3 to 4 times as large as that for fiscal 1974. The cause for this is also not clearly known. One of the conceivable causes is that pebbles which has caused deposits on the sewer, were washed away by the great flood close to the design discharge on 10th November, 1973 (Fig. 2.3) and, as a result, the basin residue load potential for 1974 seemed to be reduced. This interpretation is not unfounded, because the range of S that was obtained by the simulation of the curve for dry weather in fiscal 1973 was 6.0 to 9.7 tons and the results of simulation made by using this value of S showed a good fit to observed values. It may be added here that the observed maximum value of total load for one rainfall event was 8 tons (BOD_5) on 15th July, 1967 and on 10th November, 1973, and the field survey of pollutants source had been carried out since the beginning of 1974.

2.5 PROBLEMS AND FUTURE PROSPECT

Although several problems are left unsolved, the simulation results are satisfactory than were expected. Hereunder several problems are pointed out and future prospect is reviewed for reference' sake.

2.5.1 PROBLEMS

a. The first tough problem is that it should be remembered that the simulation is successful only for the Yabata basin. Further verification of the model must be made by applying the simulation to the other basins. Since the basin is located on a plateau, future investigation has to be made on the sewered area with old pipelines and pumping stations. Such field investigation costs much and is very difficult work. However, if a field survey of pollutant sources and two or three times of field sampling can accomplish the work, this simulation model will be greatly beneficial.

b. The difference in survey results between fiscal 1973 and 1974 poses another tough problem, which has to be checked in future. If deposition characteristics of pollutants in the basin (K and exponent m for S) should be assumed to change, the problem will become too difficult to be solved.

c. In this simulation, the component of BOD, that is, the existence of S-BOD (Soluble BOD), for instance, has not been given much attention. But there should be a model in which a part of such BOD does not contribute to the deposition and flows down. The supply from the ground surface also has to be included. It will be examined in the simulation of SS.

d. Regarding the problem that the substance of BOD may vary from dry weather to storm weather, various checkings were effected by analysing the deoxygenation coefficient, COD and TOC both for the dry and storm weather, but there were hardly any differences between them.

2.5.2 FUTURE PROSPECT

a. In this method, simulation can be carried out if the characteristics of the total basin load including S_0 and data of storm water discharge are available. The data of the discharge can be obtained from the rainfall data by using, for instance, the modified RRL method, which has been verified at the Yabata Basin and others. In such a case, simulation can be carried out with S_0, rainfall data and the formula of Qs.

b. In this sense, simulation of the prevention measures of combined sewer overflow can be carried out, by using the rainfall data for about 10 years in the past, if the formula of Qs and data concerning S_0 are given. At present, trial calculations are being made in respect to an increase in the dilution ratio and the retarding basin, etc.

c. Even if simulation for 10 years and consequently simulation for countermeasures are carried out, it is a problem to determine what extent the countermeasures should cut off the pollutants over-flow to. At present, there are neither the standards nor the criteria to determine this matter. In future, its effect upon receiving waters should be investigated.

ACKNOWLEDGEMENT

In 1967, the Sewerage section of P.W.R.I. started investigation at the Yabata Basin. In 1969, the work was succeeded by the Urban River Section. Since then, with the cooperation of many persons, a large number of results were obtained in studies concerning urban runoff and pollutants discharge. Particularly, the author is grateful to the personnels of the Planning Department of Sewerage works Bureau, Tokyo Metropolitan Government for their generous financial assistance given to him in carrying out the studies. Out of these officials, the deepest gratitude of the author goes to Mr. Nagaharu Okuno, who is now with the Sewerage Works Agency, for his kind assistance and valuable advice. This paper is dedicated to him. Beside the above, the author would like to take this opportunity of expressing his deep appreciation to all the stuff of the section who gave their assistance and advice to him in carrying out the studies.

APPENDIX

After this paper was completed, the same simulation procedure described above was applied to the other two catchment area. (A and B cities) Fig. 2.15 ~ 2.24 show results of the study.

Fig. 2.15 Simulation Result (Dry Weather)
A City, Oct. 23, 1975

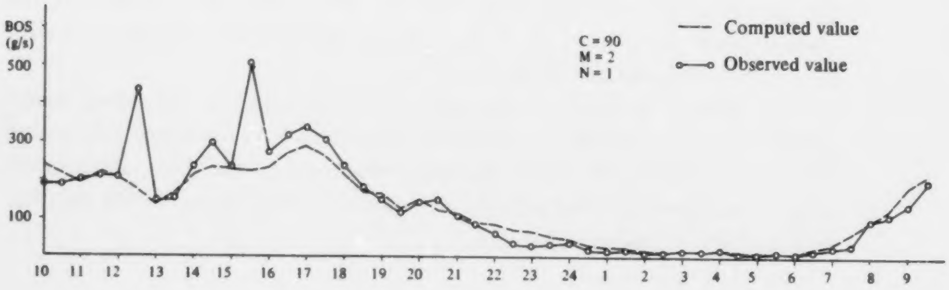


**Fig. 2.16 Simulation Result
A City, Aug. 6, 1975**

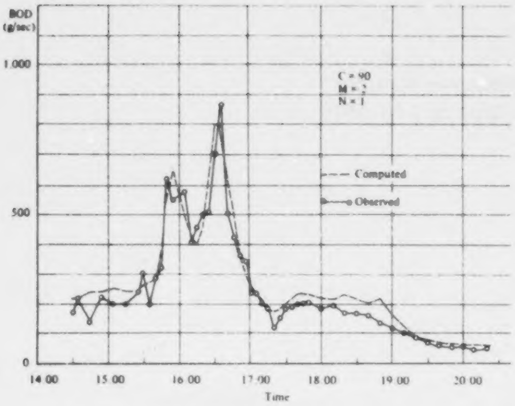


**Fig. 2.17 Simulation Result
A City, July 7, 1975**

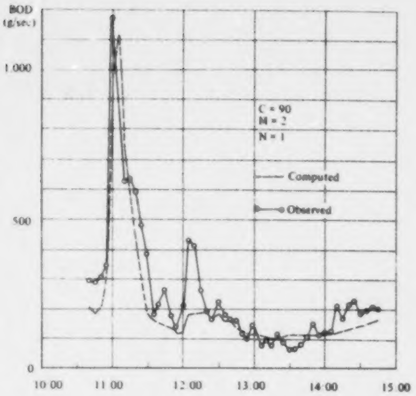


**Fig. 2.18 Simulation Result
A City, Feb. 5, 1976**

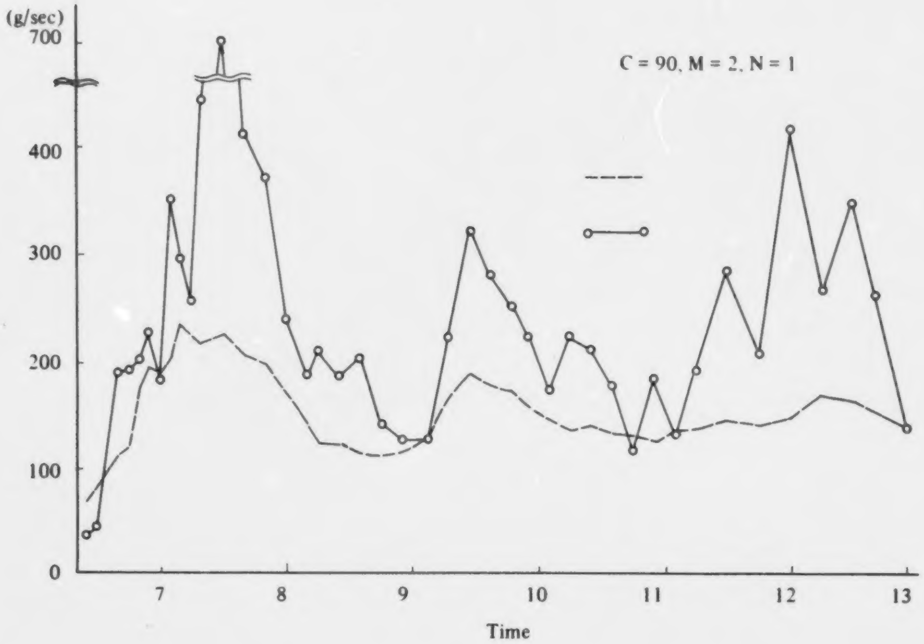


Fig. 2.19 Simulation Result
B City, July 19, 1976

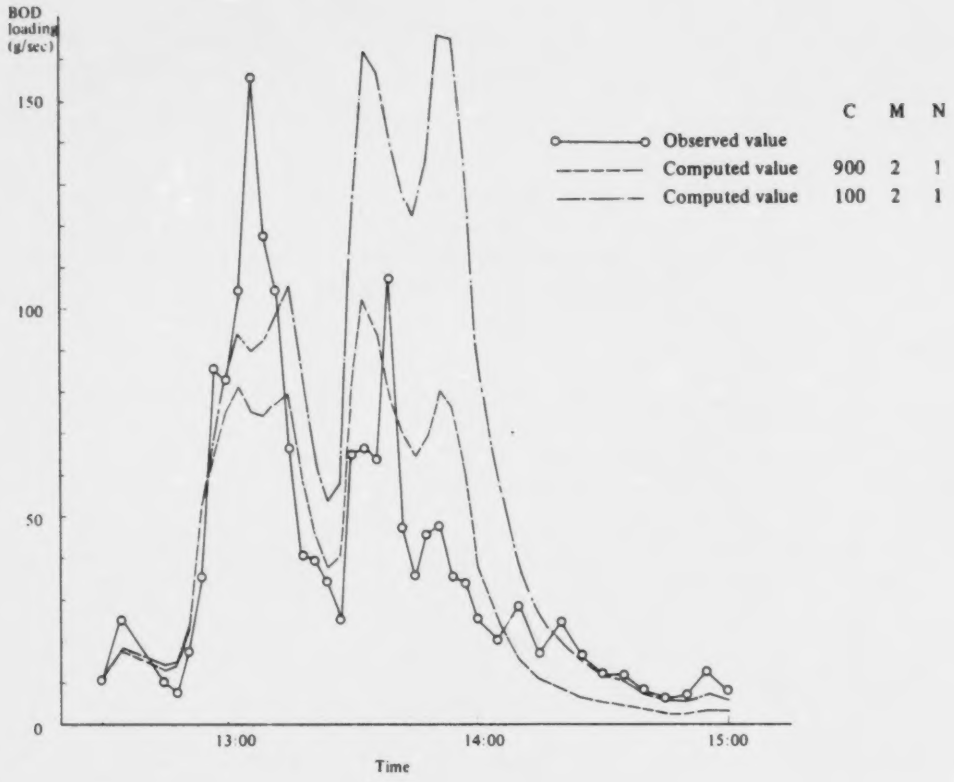
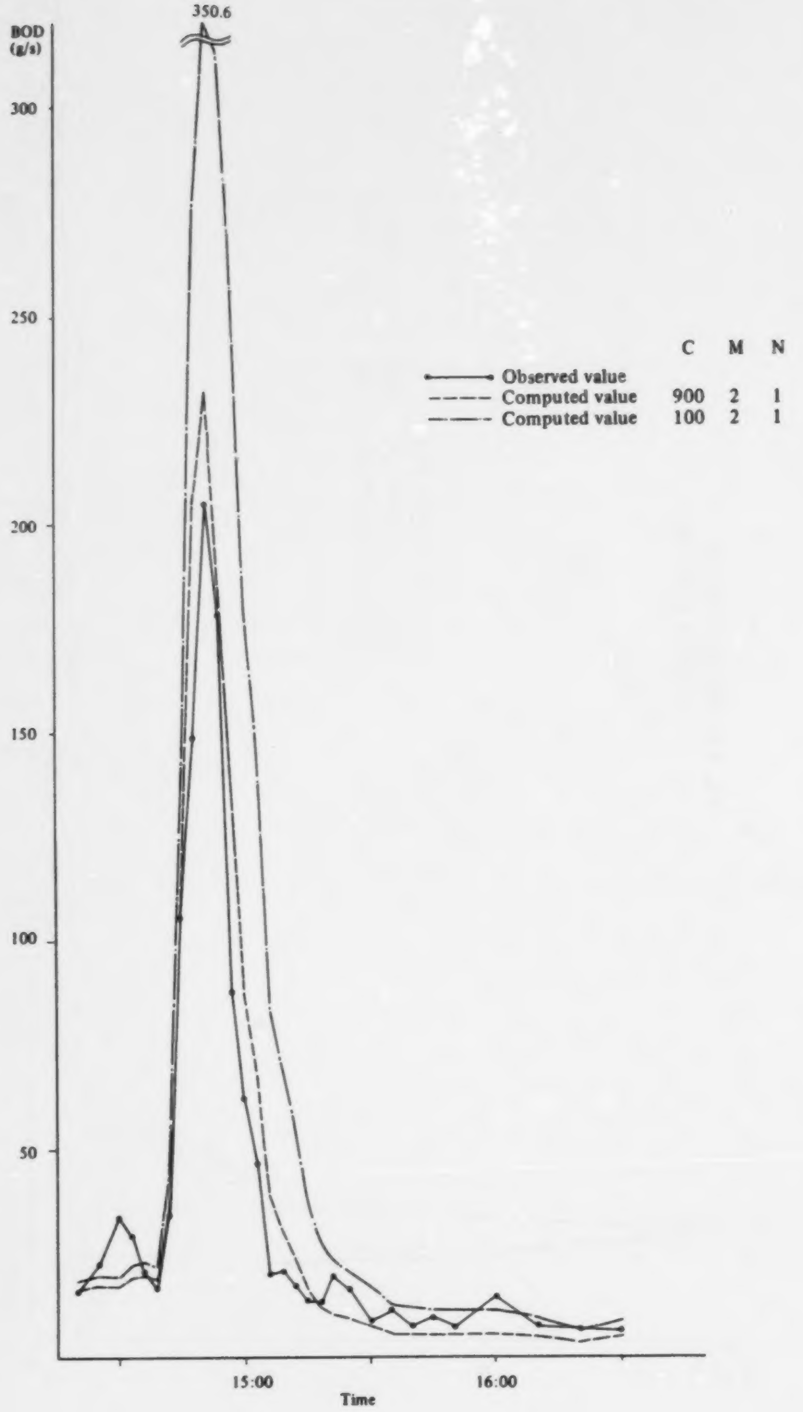


Fig. 2.20 Simulation Result
B City, Aug. 2, 1976



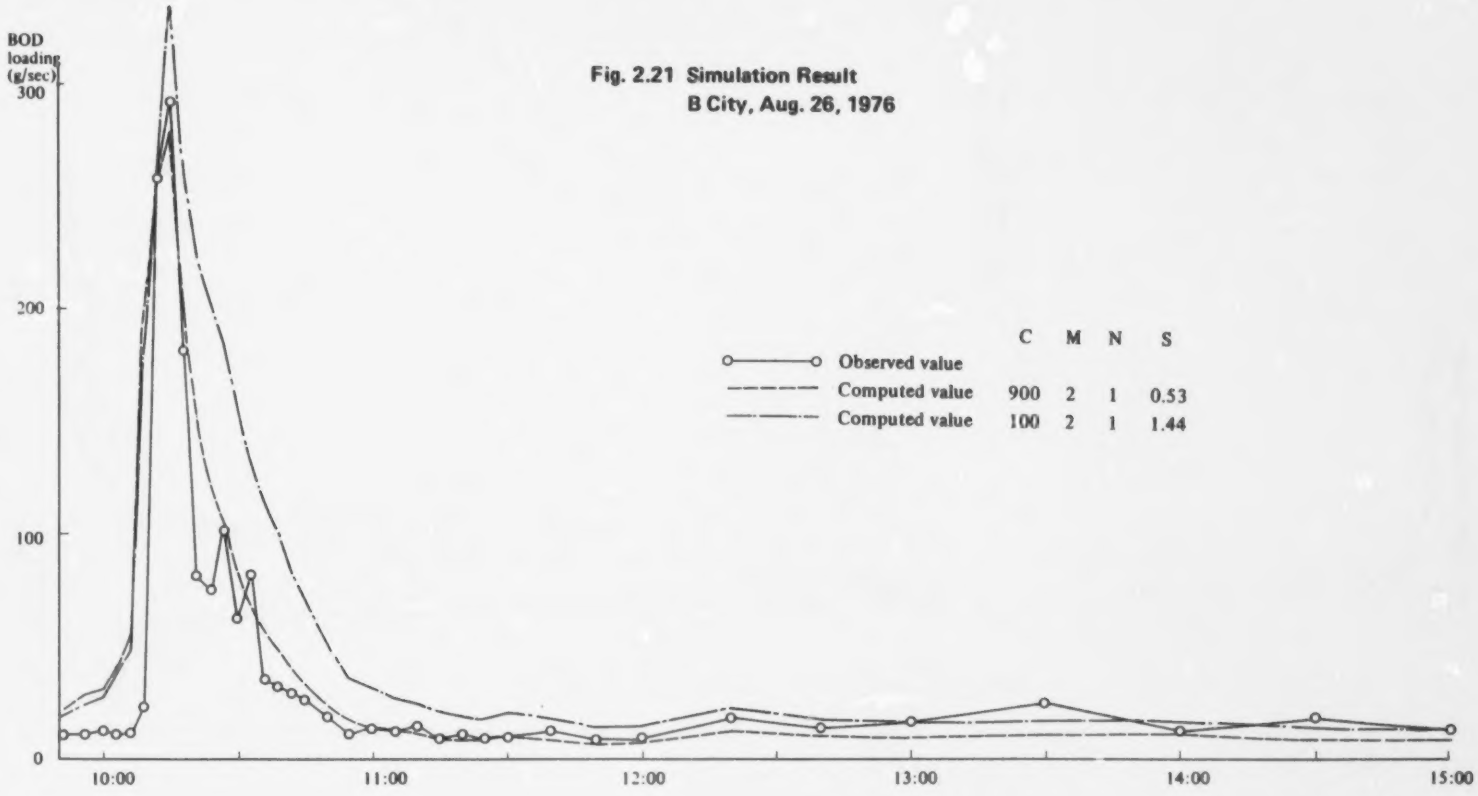


Fig. 2.21 Simulation Result
B City, Aug. 26, 1976

Fig. 2.22 Simulation Result
B City, Sept. 18, 1975

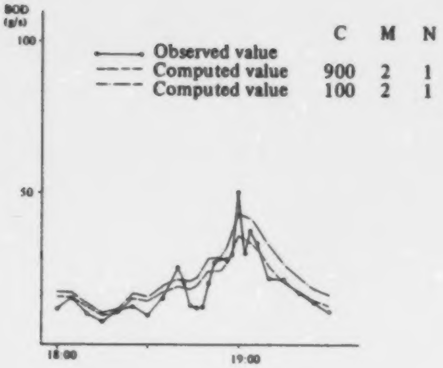


Fig. 2.23 Simulation Result
B City, Sept. 8, 1975

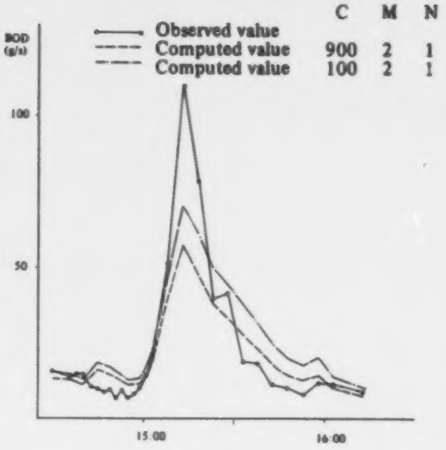
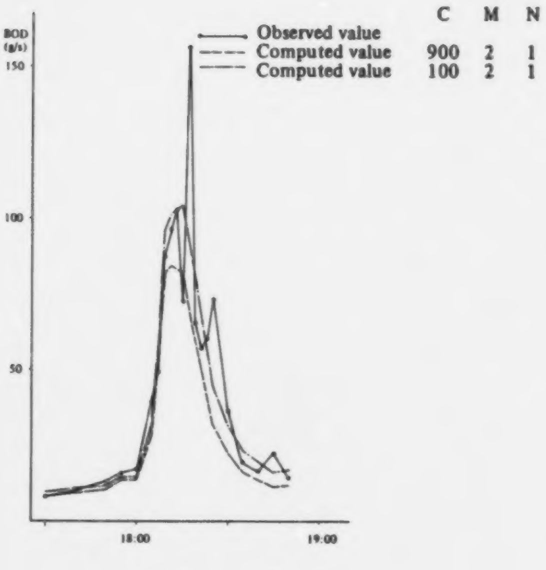


Fig. 2.24 Simulation Result
B City, Oct. 7, 1975



FIFTH US/JAPAN CONFERENCE
ON
SEWAGE TREATMENT TECHNOLOGY
PAPER NO. 3

STUDIES ON SLUDGE TREATMENT

APRIL 26-28, 1977
TOKYO, JAPAN

MINISTRY OF CONSTRUCTION
JAPANESE GOVERNMENT

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STUDIES ON SLUDGE TREATMENT

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CHAPTER 1

PERFORMANCE AND EVALUATION OF MECHANICAL SLUDGE DEWATERING FACILITIES IN YOKOHAMA CITY

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1 PERFORMANCE AND EVALUATION OF MECHANICAL SLUDGE DEWATERING FACILITIES IN YOKOHAMA CITY

According to the sewerage plan of Yokohama City, the total city area will be divided into nine treatment districts wherein ten sewage treatment plants are to be erected.

Five treatment plants are already in operation, where the secondary treatment by the activated sludge process is carried out.

As to the sludge treatment, thickened sludge (mixture of raw primary sludge and waste activated sludge) is processed by anaerobic digestion followed by dewatering in two treatment plants, by wet air oxidation followed by dewatering in a treatment plant and by dewatering of thickened sludge in other plants.

The amount of the sludge cake produced in five treatment plants is about 9,700t per annum as solid (48,150t as wet cake), 95% of which (9,160t) is disposed for land fill in municipal refuse disposal area as sludge cake and the remaining 540t is used for reclamation to agricultural land and green field as sludge cake or after mechanical sludge drying.

1.1 PRESENT STATUS OF THE MECHANICAL SLUDGE–DEWATERING FACILITIES

1.1.1 EXPLANATION OF THE FACILITIES

In city's dewatering facilities are installed belt-discharge vacuum filters (which will be referred to as BVF hereafter), pressure filters (an HPF refers to the horizontal type and a VPF the vertical type hereafter. A PF includes both of them) and centrifuges (which will be denoted an SD hereafter) as the flow sheets of sludge treatment in respective treatment plants in Figs. 1.1 ∿ 4 show. In BVFs and PFs are installed inorganic coagulant (ferric chloride, carbide slurry) dosing devices shown in Fig. 1.5 and in SDs polymeric coagulant dosing devices shown in Fig. 1.6. The types of auxiliary devices are markedly different depending upon the types of the machines as shown in flow sheets in Figs. 1.7 ∿ 10 and a list of facilities in Table 1.1. The process of installation of the dewatering facilities and the present treatment capacities in respective treatment plants are shown in Fig. 1.11.

Fig. 1.1 Flowsheet of Sludge Treatment in Chubu S.T.P.

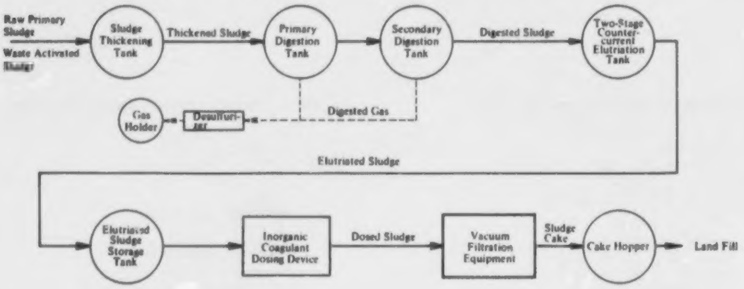


Fig. 1.2 Flowsheet of Sludge Treatment in Nambu S.T.P.

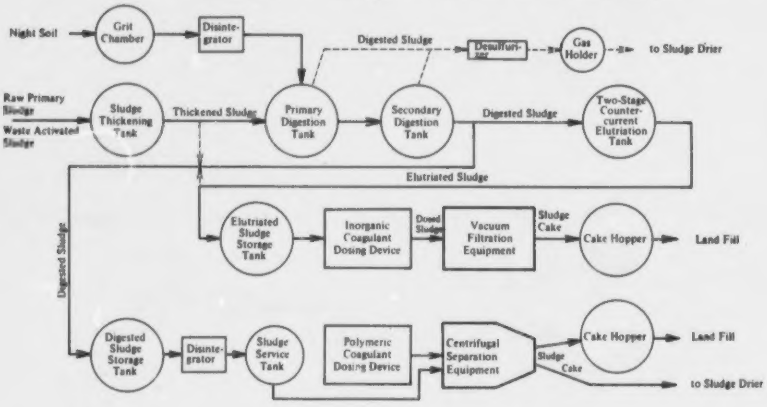


Fig. 1.3 Flowsheet of Sludge Treatment in HOKUBU S.T.P.

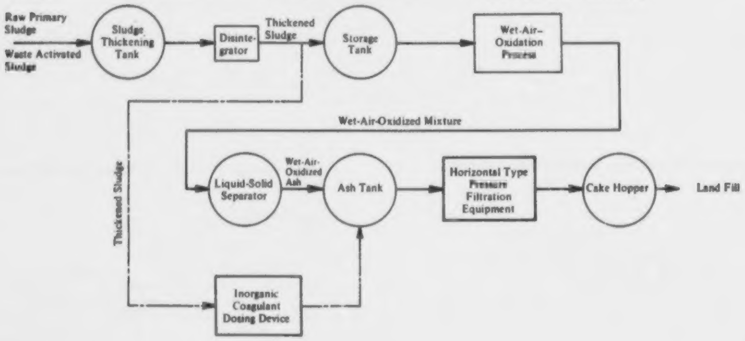


Fig. 1.4 Flowsheet of Sludge Treatment in the Second Totsuka S.T.P.

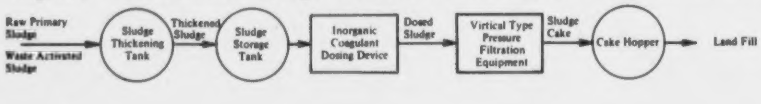
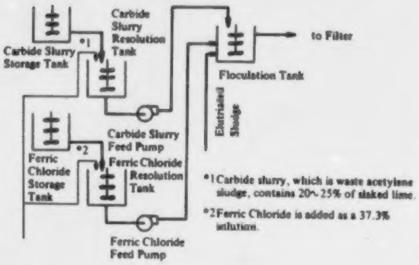


Fig. 1.5 Flowsheet of Inorganic Coagulant Dosing Device


Carbide Slurry Storage Tank
*1
Carbide Slurry Resolution Tank
to Filter
Floculation Tank
Elutriated Sludge
Carbide Slurry Feed Pump
Ferric Chloride Storage Tank
*2
Ferric Chloride Resolution Tank
Ferric Chloride Feed Pump

*1 Carbide slurry, which is waste acetylene sludge, contains 20~25% of slaked lime.

*2 Ferric Chloride is added as a 37.3% solution.

Fig. 1.6 Flowsheet of Polymeric Coagulant Dosing Device

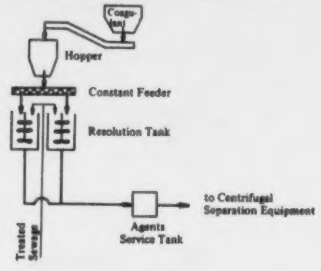


Fig. 1.7 Flowsheet of Vacuum Filtration Equipment

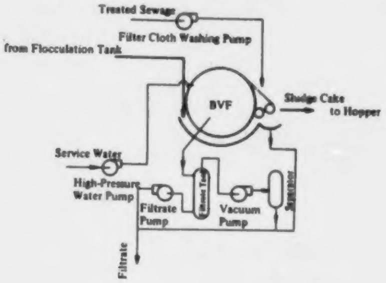


Fig. 1.8 Flowsheet of Centrifugal Separation Equipment

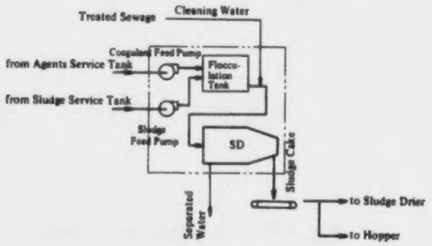


Fig. 1.9 Flowsheet of Horizontal Type Pressure Filtration Equipment

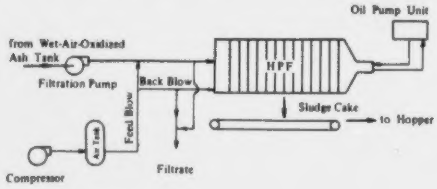


Fig. 1.10 Flowsheet of Virtical Type Pressure Filtration Equipment

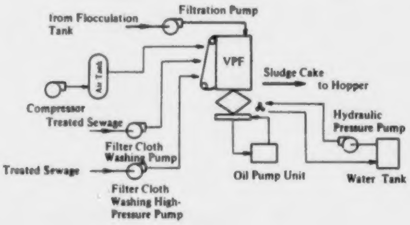
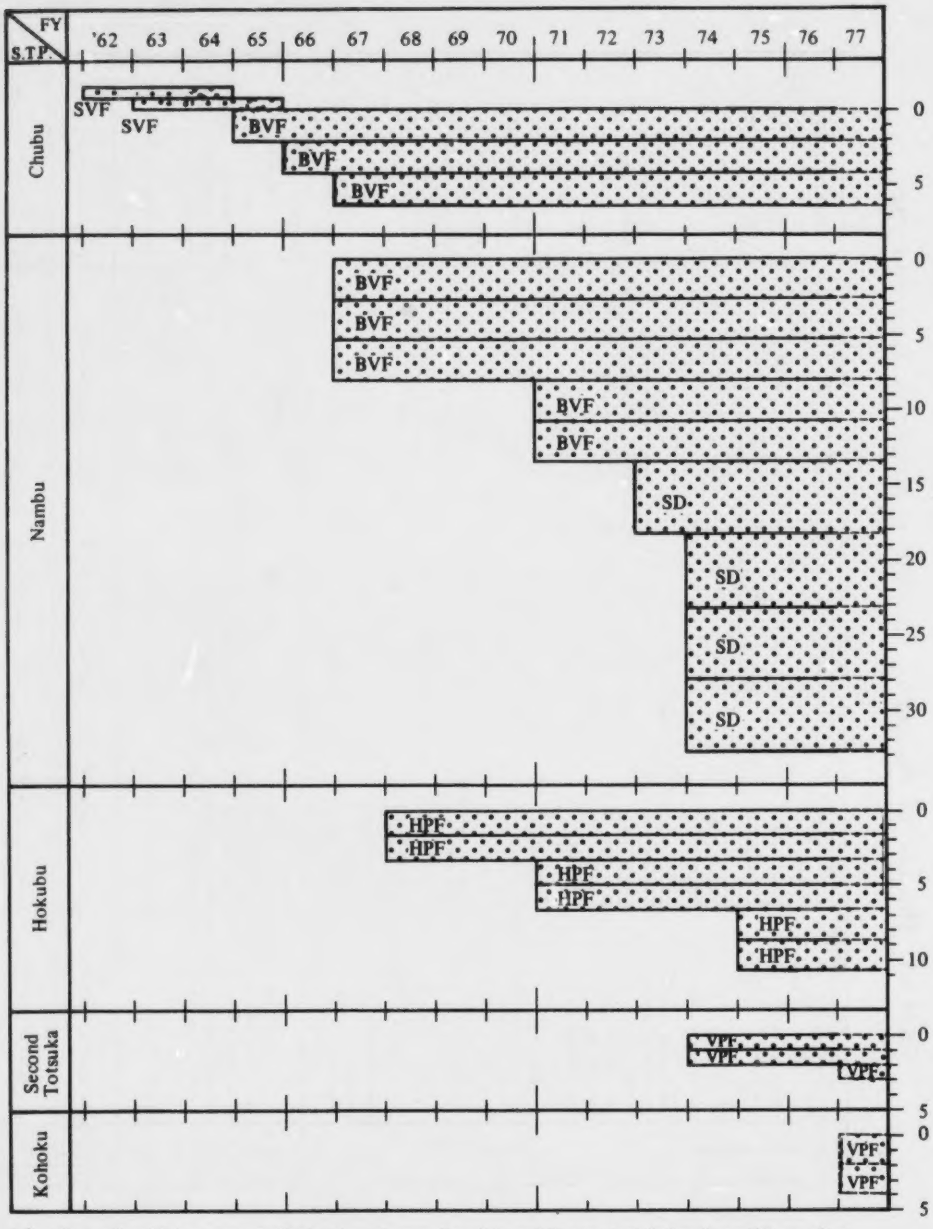


Fig. 1.11 Time Course of Installation of the Dewatering Machines in Respective Treatment Plants



* A scale on the right represents the treatment capacity of ton of the treated sludge dry solids per day. SDs are assumed to run 24 hours a day and others 7 hours a day. The working ratio is assumed to be 0.8. The treatment capacity per hour is taken after the previous results of the city.

Table 1.1 The List of Dewatering Facilities in Respective Treatment Plants

		Chubu Sewage Treatment Plant	Nambu Sewage Treatment Plant		Hokubu Sewage Treatment Plant	Second Totsuka Sewage Treatment Plant
Dewatering Machine		Type: Belt-Discharge Vacuum Filter Filtration Area: 27.1 m^2/unit Drum Dimension: 2.4 m(ϕ) x 3.5 m(W) Rotational Frequency: 0.15 - 0.5 rpm Filter Cloth: Polypropylene Number: 3 units	Type: Belt-Discharge Vacuum Filter Filtration Area: 33.9 m^2/unit Drum Dimension: 3.04 m(ϕ) x 3.5 m(W) Rotational Frequency: 0.12 - 0.5 rpm Filter Cloth: Polypropylene Number: 5 units	Type: Centrifuge (Truncated Cone Type) Flow Rate: 10 m^3/h Rotational Frequency: 2,800 rpm 2,385 rpm Number: 4 units	Type: Horizontal Type Pressure Filter Filtration Area: 83.2 m^2/unit Filtering Chamber: 50 chambers (1 m(L) x 1 m(W)) Filter Cloth: Polypropylene Number: 6 units	Type: Vertical Type Pressure Filter Filtration Area: 25 m^2/unit Filtering Chamber: 16 chambers (0.9 m(L) x 1.75 m(W)) Filter Cloth: Polypropylene Number: 2 units (to be 6)
Auxiliary Devices		(Dewatering Equipment) Vacuum Pump 3 units Filtrate Pump 3 " The Other Pumps 6 " Air Compressor 5 " Agitator 4 " (Dosing Device) Agitator for Carbide Slurry 1 unit Pump for Carbide Slurry 3 units Agitater for Ferric Chloride 2 " Pump for Ferrick Chloride 4 " Agitator for Flocculation Tank 2 " (Carrier System) Conveyer 2 " Cake Hopper 2 " Total 37 units	(Dewatering Equipment) Vacuum Pump 5 units Filtrate Pump 5 " The Other Pumps 8 " Compressor 2 " (Dosing Device) Agitator for Carbide Slurry 4 " Pump for Carbide Slurry 5 " Agitator for Ferric Chloride 2 " Pump for Ferric Chloride 3 " Agitator for Flocculation Tank 4 " (Carrier System) Conveyer 10 " Cake Hopper 5 " Total 53 units	(Dewatering Equipment) Sludge Feed Pump 4 units Coagulant Feed Pump 4 " The Other Pumps 2 Agitator 5 " (Dosing Device) Polymeric Coagulant Hopper 2 " Polymeric Coagulant Constant Conveyer 1 unit Polymeric Coagulant Constant Feeder 1 " Agitator for Resolution Tank 2 units (Carrier System) Conveyer 7 " Cake Hopper 4 " Total 32 units	(Dewatering Equipment) Filtration Pump 6 units The Other Pumps 4 " Compressor 4 " Oil Pump Unit 3 " Agitator 6 (Dosing Device) Agitator for Carbide Slurry 3 " Pump for Carbide Slurry 2 " Pump for Ferric Chloride 1 unit Agitator for Flocculation Tank 4 units (Carrier System) Conveyer 8 " Cake Hopper 2 " Total 43 units	(Dewatering Equipment) Filtration Pump 2 units The Other Pumps 8 " Compressor 2 " Oil Pump Unit 1 unit (Dosing Device) Agitator for Carbide Slurry 2 " Pump for Carbide Slurry 2 " Agitator for Ferric Chloride 2 " Pump for Ferric Chloride 6 " (Carrier System) Conveyer 5 " Cake Hopper 4 " Total 34 units
Sludges		Elutriated	Elutriated	Digested	Wet-Air-Oxidized (Ash) Thickened	Thickened
Electric Installation		Actual Capacity 179.95 kW Installed Capacity 205.35 kW	Actual Capacity 319.5 kW Installed Capacity 365.2 kW	Actual Capacity 114.25 kW Installed Capacity 125.25 kW	Actual Capacity 240.65 kW Installed Capacity 273.65 kW	Actual Capacity 149 kW Installed Capacity 280 kW
Operators		4 persons for 3 units	4 persons for 5 units	1 person for 4 units	(During Wet-Air Oxidation) 1 person for 6 units During Dewatering of Thickened Sludge 3 persons for 6 units	2 persons for 2 units
Installation Area	Dewatering Machine Cabins	(34.8 m x 14 m) 487.2 m^2	(27.0 m x 15 m) 405 m^2	(27.0 m x 15.0 m) 405 m^2	(28 m x 13.6) 380.8 m^2	(15 m x 25 m) 375 m^2
	Dewatering Machines	(4.0 m x 5.5 m) 22 m^2/unit	(4.9 m x 5.5 m) 26.95 m^2/unit	(2.5 m x 4 m) 12 m^2/unit	(6.3 m x 1.5 m) 9.45 m^2/unit	(4.5 m x 2.5 m) 11.25 m^2/unit

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1.1.2 THE REASON OF SELECTING THE MACHINES

a. Chubu Sewage Treatment Plant

Chubu Sewage Treatment Plant, the first treatment plant erected in Yokohama City, has been in operation since 1962. In selecting the dewatering facilities, scraper-discharge vacuum filters whose positive achievement was known then were adopted.

Later, the machines were changed to BVFs in 1965 because the achievement of the BVF with a larger treating capacity than the scraper-discharge vacuum filter was known.

b. Nambu Sewage Treatment Plant

BVFs were installed as the dewatering machines of the Nambu Sewage Treatment Plant in 1967 after the experience in the Chubu Sewage Treatment Plant. However, as the ratio between the night soil and the sewage sludge decreased later, the expected capacity could not be achieved and the enlargement of the facilities became necessary. After many considerations, SDs with improved efficiency by structural modifications and the use of polymeric coagulant were adopted in 1973. These machines fulfilled the requirements of limited installation space and continuous operation with infrequent inspection.

c. Hokubu Sewage Treatment Plant

For dewatering of the sludge after wet air oxidation obtained in Hokubu Sewage Treatment plant, BVFs and HPFs were compared and HPFs which yield cakes with lower water content were installed in 1968.

d. Second Totsuka Sewage Treatment Plant

PFs were selected as the dewatering machines of Totsuka Sewage Treatment Plant, which was expected to yield cakes with lower water content so that the disposal by land fill is easy and the disposal after incineration expected in the future is advantageous. VPFs with a greater filtration rate and a smaller installation space than HPFs were installed and have been in operation since 1974.

1.1.3 MAINTENANCE

a. Characteristics of Maintenance

Various running characteristics, problems and periodical inspections etc. are shown in Table 1.2.

b. Main Modifications

Various modifications have been carried out after our operating experience, some examples of which will be described below.

– BVF

– To prevent clogging of the filter cloth and to improve the efficiency of the dewatering machines, automatic filter cloth washing machines which utilize water at high pressure (rapidly filtered water after secondary treatment) were installed. Washing is carried out automatically for about 30 minutes after the dewatering operation is over by spraying water at high pressure evenly over the filter cloth as the washing nozzle reciprocates horizontally while the drum rotates. As a result of this modification, the dewatering operation could be prolonged by about an hour a day.

Table 1.2 Characteristics of Maintenance

Items \ Types of Machines	BVF	PF	SD
Operating Characteristics	(1) Because the dewatering is conducted by filtering by vacuum, there is a limit to the water content of the sludge cake. (2) Not suited for dilute sludge. Empirically, when TS concentration is less than 3%, the cake layer is not formed, the filter cloth cloggs quickly, the cake barely comes off and hence dewatering is impossible. (3) If the disposal of the cake by land fill is planned or if continuously stable cake is needed, the feed sludge with more than 4% of TS is necessary. (Empirical rule) (4) To maintain the filtering capacity, large amount of the filter cloth and the washing water is necessary. The filter cloth must be washed with acid for regeneration. (5) Carbide slurry used as a dewatering adjutant adheres to the vacuum tube in the drum as hard scales lowering the filtration rate and hence must be cleaned periodically. (6) The filter cloth tends to form wrinkling which influences the filtration rate. (7) Because the open area is large, some measure must be taken to prevent odor.	(1) Dewatering is conducted by pressing and squeezing, and hence the water content of the sludge cake can be low. (Sludge cake with the lower water content can be obtained than any other types of the machines.) (2) Since the machine is operated batchwise, when the initial TS concentration is higher, the filtration rate is greater and more stable. (3) Not suited for dilute sludge. Dewatering is not impossible but the cycle time must be considerably prolonged, the dosing rate must be extremely large, and the filtration rate is expected to become very low. (4) The cycle time can be controlled but it will not improve the filtration rate to any great extent. (5) To maintain the filtering efficiency, filter cloth washing water at high pressure is necessary. The filter cloth must be washed with acid for regeneration. (6) If the dewatering process is carried out while the cake is adhered to the frame of the filter cloth, it sometimes leaks between the filter cloth and filter plate.	(1) Dewatering mechanism is the forced consolidation and precipitation. Therefore, it is possible to dewater at a low concentration of TS. If one may ignore the recovery ratio of solids to some extent, the machine may be operated without dosing. (2) If a proper coagulant is selected, the machine may be operated at a low dosing rate. The water content of the cake is influenced by the amount of VTS. (3) The feeding rate of dry solids is limited by the strength of the gears. Hence, if the sludge is very thick, the flow rate must be controlled. (4) Properties of the sludge cake are not suited for disposal by land fill. But the water content in the sludge cake is small in comparison with BVFs. Stable production of the cake is not limited by the concentration of the feed sludge in contrast to other types of machines. (5) Feed sludge rich in sand wears the screw rotor and the outer drum quickly. Hence the sand must be fully removed. (6) Rotation at high speed causes vibration and noise when the machine is in operation. (7) Because of the high-speed rotation, screw rotor, outer drum and the outlet tend to wear quickly and must be periodically repaired. (8) The separated liquid foam readily sometimes making drainage difficult.
Details of the Periodical Inspection	(1) No legal obligation of inspection. (2) Auxiliary devices such as a vacuum pump or a filtrate pump are important in dewatering and require rigorous periodical repair. (3) Machines with sliding parts such as a cake discharge roller, a high flow valve and a compressor must be periodically inspected. (4) The filter cloth must be washed with acid for regeneration.	(1) No legal obligation of inspection. (2) The filter plate moving mechanism must be periodically repaired. Adjustment and renewal of parts of the oil cylinder or the opening and closing mechanism of the filter plate are necessary. (3) Machines with sliding parts must be periodically inspected. (4) Same as BVF (4)	(1) Periodical and voluntary inspection of a centrifuge with a revolving part at high speed is imposed by the Safety and Hygiene in Labor Act. (2) Because of the high-speed rotation, planet gears and bearings must be periodically inspected. (3) Wear of the parts which slide or collide with the sludge such as screw rotor and the outer drum must be repaired.
Maintenance	(1) Duration of the filter cloth must be judged and the filter cloth must be exchanged. (2) Coming off of the cake, winding of the filter cloth etc. must be inspected. If necessary the revolving rate of the drum and the dosing rate must be controlled. (3) There are many kinds of auxiliary devices such as a vacuum pump, and their maintenance is indispensable for stable and effective dewatering.	(1) Same as BVF (1) (2) The wrinkling of the filter cloth, the leakage between the filter cloth and the filter plate, coming off of the cake and so forth must be watched. Cycle time must be adjusted. (3) Careful maintenance of the auxiliary devices is indispensable for stable and effective dewatering.	(1) The separated liquid, the state of the cake and driving force of the rotor must watched. The dosing rate and the feeding rate must be controlled.
Working environment	(1) While washing the filter cloth, the worker might inhale the volatile component of acid. (2) Dewatering of the thickened sludge causes bad odor.	Same as BVF	(1) Noise and vibration during operation are large.

– To better the coming off of the cakes, flapper rollers were installed. By this modification, TS concentration limit at which the coming off of the cakes becomes bad was improved to 3% from 3.5%.

– SD

– Because the inner drum screw and solids discharge outlets wore out rapidly, the padding material was changed to a wear-proof one. By this modification, the life time was lengthened from 1,800 hours to over 3,000 hours. For this reason and for having changed the rate of rotation from 3,400 rpm to 2,400 rpm, the machines have already been operating continuously over 4,000 hours.

– HPF

– When slack filter cloth is introduced into plate closing process, wrinkling of the filter cloth arises which results in breakage. To prevent this, a weight (a round bar of vinyl chloride) was placed at the lower end of the filter cloth, by which modification the wrinkling arose no more.

1.1.4 THE RESULTS OF OPERATION

The results of operation in respective treatment plants in 1975 are shown in Tables 1.3 ∿ 6. Because the SDs in Nambu Sewage Treatment Plant were modified, the results after June, 1976 are shown (Table 1.4).

In Table 1.7 operating states of respective dewatering machines are compared. The dewatering rates of all of the dewatering machines were lower than was initially expected partially in connection with the quality of the feed sludge.

The difference between the standard operation and maintenance cost and the actual one arose mainly due to the difference in the number of the operating personnel.

Fig. 1.12 Automatic Filter Cloth Washing Device by High-Pressure Water

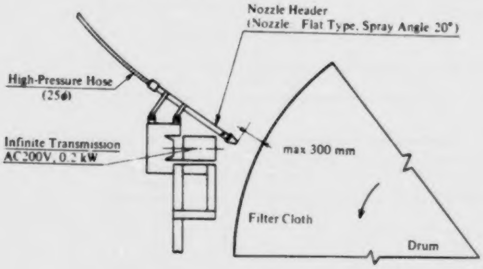


Fig. 1.13 Cake Flapper Device

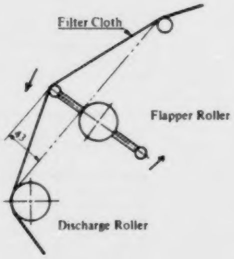


Table 1.3 The Results of Operation at Chubu S.T.P.

(BVF)

Month, Year	Feed Sludge (Elutriated Sludge)		Dosing Rates(%) *1		Sludge Cake TS (%)	Filtration Rate (Kg/m²·h) *2	Average Operating Time (h/unit·day) *3
	TS(%)	VTS(%)	Carbide Slurry	Ferric Chloride			
Apr., 1975	3.8	49	32.9	5.9	22	13.0	3.0
May,	3.0	49	50.8	10.5	21	9.4	3.5
Jun.,	3.9	46	54.0	11.0	22	9.9	3.4
Jul.,	4.6	44	42.3	8.2	24	12.9	4.3
Aug.,	4.0	47	44.2	8.2	25	12.8	3.7
Sept.,	4.1	48	47.3	8.3	22	11.5	3.0
Oct.,	4.1	46	40.9	7.9	23	12.8	4.9
Nov.,	4.9	48	34.1	6.9	26	15.6	3.3
Dec.,	5.8	38	27.9	5.1	24	19.2	3.8
Jan., 1976	5.0	43	31.2	6.4	24	19.9	3.7
Feb.,	4.0	48	44.0	8.9	24	13.9	3.7
Mar.,	4.3	49	37.1	7.9	23	15.6	2.8
Average	4.3	46	40.6	7.9	23	13.9	3.5

*1 The dosing rates are the weight ratios of slaked lime and ferric chloride to the amount of solid in the sludge. (Same for all of the treatment plants.)

*2 The filtration rate is given on the basis of the amount of solids in sludge cake excluding the coagulant. (Same for all of the treatment plants.)

*3 The average operating time is the total operating time divided by the number of machines operated. (Same for all of the treatment plants.)

Table 1.4 The Results of Operation at Nambu S.T.P.

(BVF)

Month, Year	Feed Sludge (Elutriated Sludge)		Dosing Rates(%)		Sludge Cake TS (%)	Filtration Rate (Kg/m²·h)	Average Operating Time (h/unit·day)
	TS(%)	VTS(%)	Carbide Slurry	Ferric Chloride			
Jun., 1976	3.4	45	39	7.8	22	10.0	6.0
Jul.,	3.8	41	33	6.7	23	8.8	5.7
Aug.,	3.6	40	26	6.4	23	8.2	6.3
Sept.,	3.7	42	28	6.6	22	10.4	5.8
Oct.,	4.6	41	25	5.2	25	11.5	5.6
Nov.,	4.5	45	24	5.5	22	9.8	3.4
Average	3.9	42	29	6.4	23	9.8	5.5

(SD)

Month, Year	Feed Sludge (Digested Sludge)		Dosing Rates(%)	Sludge Cake TS (%)	Feeding Rate of Dry Solids (Kg/h) *	Average Operating Time (h/unit·day)
	TS(%)	VTS(%)	Polymeric Coagulant			
Jun., 1976	3.2	44	0.86	25	284	20.1
Jul.,	3.6	43	0.83	26	278	17.3
Aug.,	3.8	43	0.78	25	291	13.2
Sept.,	3.5	43	0.78	26	277	15.0
Oct.,	3.4	44	0.78	26	246	16.0
Nov.,	3.1	47	0.74	23	260	11.1
Average	3.4	44	0.80	25	273	15.5

* The dewatering Rate of the SD is represented by the feeding rate of dry solids. The rated capacity of this centrifuge is 10 m^3/h.

Table 1.5 The Results of Operation at Hokubu S.T.P.

(HPF)

Month Day, Year	Feed Sludge (Thickened Sludge)		Dosing Rates(%)		Sludge Cake TS (%)	Filtration Rate $(Kg/m^2 \cdot h)$	Average Operating Time (h/unit·day)
	TS(%)	VTS(%)	Carbide Slurry	Ferric Chloride			
May 2, 1975	9.2	38	38	5.0	41	3.5	5.3
Jul.25,	7.3	36	54	3.6	36	2.8	3.6
Jul.31,	5.6	36	34	4.0	37	2.7	
Oct.22,	7.9	-	34	2.4	42	4.1	3.5
Oct.23,	7.9	-	46	2.5	40	3.9	
Mar.5, 1976	10.2	-	59	5.0	44	3.5	3.5
Mar.9,	9.8	-	59	6.0	41	3.0	
Mar.16,	10.4	-	76	6.0	44	3.0	
Mar.26,	-	-	65	5.0	37	2.8	
Average	8.5	37	52	4.4	40	3.3	4.0

Table 1.6 The Results of Operation at the Second Totsuka S.T.P.

(VPF)

Month, Year	Feed Sludge (Thickened Sludge)		Dosing Rates		Sludge Cake TS (%)	Filtration Rate $(Kg/m^2 \cdot h)$	Average Operating Time (h/unit·day)
	TS(%)	VTS(%)	Carbide Slurry	Ferric Chloride			
Apr., 1975	6.0	39	53	8.9	43	8.0	8.3*1
May,	6.2	40	39	9.0	38	7.5	7.1
Jun.,	7.3	37	51	7.7	40	8.0	6.9
Jul.,	7.3	34	37	7.8	44	9.7	6.9
Aug.,	5.6	40	39	7.3	45	7.8	7.0
Sept.,	5.0	45	39	7.3	44	6.7	6.6
Oct.,	7.9	34	38	7.2	44	11.0	6.6
Nov.,	5.8	43	42	7.9	42	7.4	6.7
Dec.,	4.5	53	51	12.0	35	6.1	6.5
Jan., 1976	3.0	61	56	13.0	31	4.9	10.6*2
Feb.,	3.5	62	59	11.0	31	3.7	15.6*2
Mar.,	4.6	58	56	11.0	34	4.6	7.2
Average	5.5	46	47	9.1	39	7.1	8.0

*1 In April, 1975, the dewatering machines were operated for 10 hours a day.

*2 From Jan. 28 to Feb. 21, 1976, the machines were operated for 24 hours a day.

Table 1.7 Comparison of Respective Dewatering Machines

Items	S.T.P. / Types of Machines	Chubu	Nambu		Hokubu	Second Totsuka
		BVF	BVF	SD	HPF	VPF
Feed Sludge (annual averages)	Kinds	Elutriated Sludge	Elutriated Sludge	Digested Sludge	Thickened Sludge	Thickened Sludge
	TS(%)	4.3	3.9	3.4	8.4	5.5
	VTS(%)	48.0	42.0	44.0	37.0	45.5
Dosing Rates (annual averages)	Carbide Slurry(%)	40.6	29.0	· Polymeric Coagulant 0.8	51.7	46.6
	Ferric Chloride(%)	7.9	6.8	-	4.4	9.1
Sludge Cake (annual average)	TS(%)	23.0	23.0	25.0	40.2	39.2
Dewatering Rates (kg/m²·h)	Rated	25.0	25.0	The feeding rate of Dry Solids 400 kg/hr	5.5	15.0
	Actual	13.9	9.8	The feeding rate of Dry Solids 273 kg/hr	3.3	7.1
Actual Working Times of Dewatering Machines (h/unit·day)	Actual	3.5	5.5	15.5	4.0	8.0
	Standard*	5.6	5.6	19.2	5.6	5.6
Operators (persons/unit)	Actual	1.3	0.8	0.25	0.5	1.0
	Standard*	0.8	0.8	0.2	0.4	0.4
Operation and Maintenance Costs per ton of Feed Dry Solids ($/ton)	Actual	170.14	138.42	104.73	106.45	133.39
	Standard*	148.61	138.59	104.04	99.97	119.01

* These are the estimated standard values of the working hour and the operating personnel for respective machines.

The standard cost was calculated on this basis.

Table 1.8 Details of the Standard Operation and Maintenance Cost per Ton of Feed Dry Solids

Types of Machines / Details	BVF (Chubu)		BVF (Nambu)		SD		HPF		VPF	
	Feed Dry Solids 6.33t/day Number 3 units		Feed Dry Solids 9.30t/day Number 5 units		Feed Dry Solids 26.11t/day Number 4 units		Feed Dry Solids 9.23t/day Number 6 units		Feed Dry Solids 1.99t/day Number 2 units	
	Costs		Costs		Costs		Costs		Costs	
	Sums($)	Percentage	Sums($)	Percentage	Sums($)	Percentage	Sums($)	Percentage	Sums($)	Percentage
Coagulant Cost	14.14[*1]	(10)	10.10	(7)	34.67[*3]	33	17.97	(18)	16.02	(14)
	19.97[*2]	(13) 23	17.17	(12) 19			11.11	(11) 29	22.68	(19) 33
Personnel Cost	12.84	9	14.58	11	1.05	1	8.81	9	13.62	11
Electric Cost	3.87	2	3.67	3	6.08	6	1.46	1	6.57	6
Carriage Cost	17.94	13	18.21	13	12.44	12	12.04	12	12.25	10
Disposal Cost	75.37	51	68.83	50	47.02	45	45.50	46	46.32	39
Repair Cost	2.48	2	6.03	4	2.78	2	3.08	3	1.55	1
Grand Total	148.61	100	138.59	100	104.04	100	99.97	100	119.01	100

[*1] Carbide Slurry [*2] Ferric Chloride [*3] Polymeric Coagulant

1.1.5 OPERATION AND MAINTENANCE COST

Details of the standard operation and maintenance cost are shown in Table 1.8.

a. The BVF is the most expensive and the HPF the least expensive. This is considered to be related to the fact that the feed sludge to the BVF (TS 4.3%, VTS 48%) is the most difficult to dewater while the feed sludge to the HPF (TS 8.4%, VTS 37%) is easy to dewater.

b. The cost of disposal occupies a large portion of the cost, between 39% and 51%, in all of the machines. Then follows the coagulant cost which is between 23% and 33%. The fact indicates that, to lessen the dewatering cost, a machine with a smaller amount of cake produced, i.e., with lower water content of the sludge cake, and a lower dosing rate should be selected.

1.2 THE RELATION BETWEEN THE FEED SLUDGE AND THE DEWATERING EFFICIENCY

The major problem in dewatering treatment of the sludge is the fluctuation in quantity and quality of the feed sludge. The fluctuation in quantity can be coped with by storage and proper selection of the operating hours.

The fluctuation in quality of the sludge causes the fluctuation in the water content of the sludge cake resulting in poor coming off of the cake and extreme diffuculty of the dewatering operation.

Sludge treatment facilities have hitherto been designed by application of the standard dosing rate and filtration rate expecting sludge of the standard quality. Therefore, when the dewatering of the sludge becomes poor, the dosing rate must be increased and the operating time must be temporarily prolonged. Such changes not only cause problems in sludge treatment and personnel management, but also induce the suppression of the withdrawal of the sludge and the increase in the amount of the circulating sludge, which together cause serious problems in the quality of the sewage treatment plant effluent.

Therefore in design, operation management and evaluation of the dewatering machines, it is important to know the characteristics of the machines, such as the quality of the product of the machines, the rate of production and the controlling factors, in response to the fluctuation in the quality of the feed sludge.

The dewatering process has a three dimensional structure with variables, the quality of the feed sludge (x). water content of the sludge cake (y) and the dewatering rate (z) and the controlling factor (p) as the parameter. However, it is not easy to solve it. Therefore, as an approximation for practical purposes, the relations, $y=f(x)$, $z=f(x)$ and $p=f(x)$ with the assumptions that x is a given variable and that two excluded variables are within appropriate ranges, provide enough information to decide how to cope with the situation in the plant and to reveal the characteristics of the dewatering machines.

Among many factors which may be taken to represent x,y,z and p, our operating experience up to now suggests to select VTS for x, dosing rate for p, water content of the sludge cake for y and the filtration rate on the basis of the treated solid for z. Taking such factors as the variables, the aforementioned relations will be as follows.

1.2.1 TS AND VTS

TS of the feed sludge has hitherto been counted among the factors which determine the dewatering efficiency of the dewatering machines including the filtration rate. It is also our experience that the dewatering efficiency changes depending upon the TS. But when we encounter seasonal difficulty in dewatering of the sludges, TS decreases but at the same time VTS increases. The phenomenon is thought to arise from the facts that the feed sludge has been thickened and separated in a so-called gravity thickener and that the components of VTS are inhibitory factors of thickening with high viscosity and strong affinity to water. Therefore, as a parameter to represent the properties of the sludge VTS is preferred.

The relation between TS and VTS is shown in Fig. 1.14.

The seasonal fluctuation in the amount of VTS is shown in Fig. 1.15.

The amount of VTS fluctuates seasonally in any of the municipal treatment plants. When the VTS level is high, dewatering is difficult. Also, the longer the plant has been in operation, the higher the VTS level is.

1.2.2 DEWATERING RATE AND VTS

The relation between the filtration rate and VTS in the BVF and the VPF is shown in Fig. 1.16. The straight line in the figure is a regression line.

In the information obtained from the BVF, the range of the VTS level of the elutriated sludge is narrow and scattering is large. Therefore, the correlation between VTS and the filtration rate is more marked in the VPF than in the BVF.

Some of the characteristics of the machines found in this figure are as follows.

a. Because BVFs are run continuously, the filtration rate of the BVF is greater than that of the VPF, but the VTS level of the feed sludge seems to influence the performance of BVFs to a greater extent than VPFs. It has been experienced that, at a higher level of VTS, the cake layer becomes thinner, comes off the filter cloth less readily and, along with the clogging of the filter cloth, makes it unfeasible to filter.

b. VPFs are run batchwise in a cycle comprising filtration, squeezing and cake discharge because of its mechanism. Therefore, the filtration rate is lower than that of the BVF but VTS influences to a smaller extent than the BVF.

c. In contrast to BVFs and PFs, SDs dewater by forced consolidation with the centrifugal force. Therefore, to express its capacity, feeding rate of dry solids (flow rate × TS) is used. If the flow rate more than the rated capacity is supplied when TS is low, the holding time shortens and hence the recovery ratio becomes bad. In practice, the feeding rate of dry solids is about 80% of the rated capacity.

1.2.3 DOSING RATE AND VTS

In Figs. 1.17, 18 is shown the fluctuation in the dosing rate of ferric chloride and carbide slurry depending upon VTS during dewatering of the elutriated sludge by the BVF.

In Figs. 1.19, 20 is shown a similar fluctuation during dewatering of the thickened sludge by the VPF.

In Fig. 1.21 is shown a fluctuation in the dosing rate of the polymeric coagulant depending upon VTS during dewatering by the SD.

The present method of dosing control is instrumentally a proportional adding method to the feed dry solids, but in practice the operator determines the appropriate level by observations of the cake coming off the filter cloth during actual operation. Hence, the scattering in the dosing rate is large, but it may be assumed that the change in the dosing rate depending upon VTS during dewatering of the thickened sludge by the VPF is considerable.

In the case of the elutriated sludge, the correlation is small on the same reason as the aforementioned filtration rate. The dosing rate for the elutriated sludge is supposed to be smaller than the thickened sludge. The dosing rate of the polymeric coagulant used in SDs is also influenced by VTS.

1.2.4 WATER CONTENT AND VTS

In Figs. 1.22, 23 are shown relations between the water content of the sludge cake and VTS for respective dewatering machines.

Water content of the sludge cake is also influenced by VTS. Water content of the cake is the highest in the BVF. The performance of the SD is most drastically influenced by VTS, and at a higher level of VTS, the water content of the cake is about the same as that of the BVF.

Fig. 1.14 TS vs. VTS

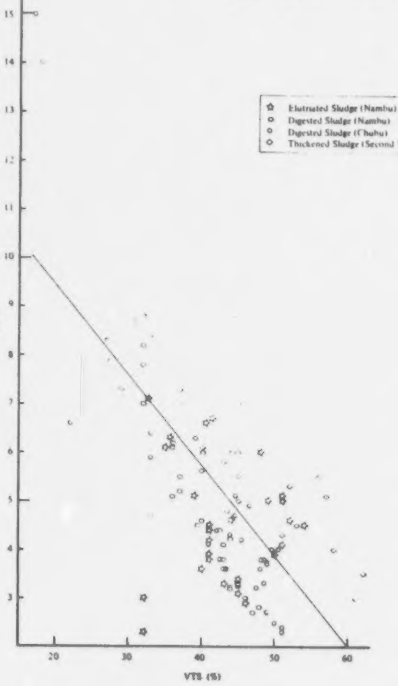


Fig. 1.15 Seasonal Fluctuation of VTS of Thickened Sludge

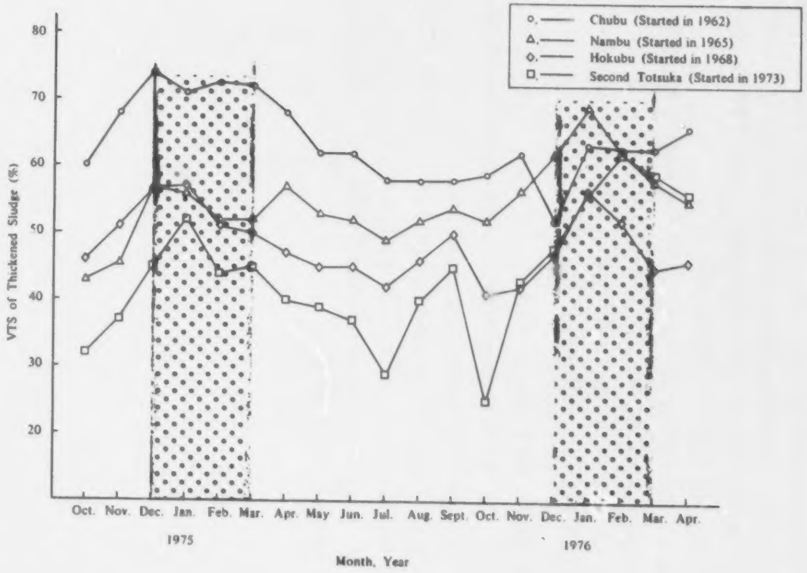


Fig. 1.16 Filtration Rate vs. VTS

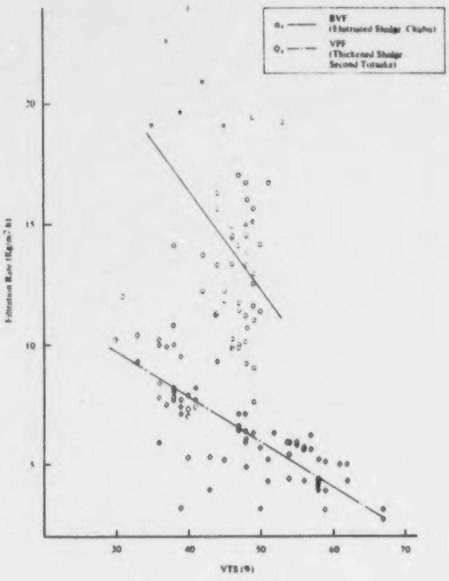


Fig. 1.17 Dosing Rate (Ferric Chloride) vs. VTS

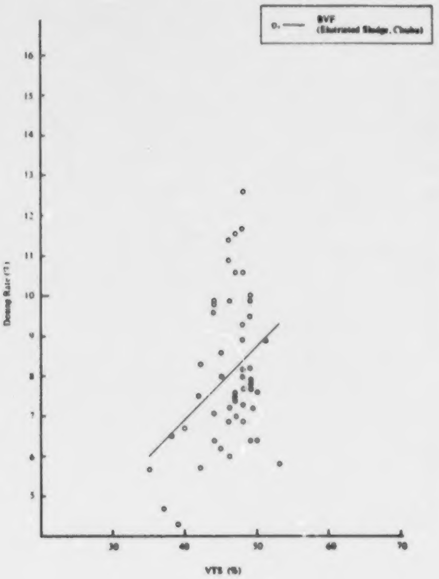


Fig. 1.18 Dosing Rate (Carbide Slurry) vs. VTS

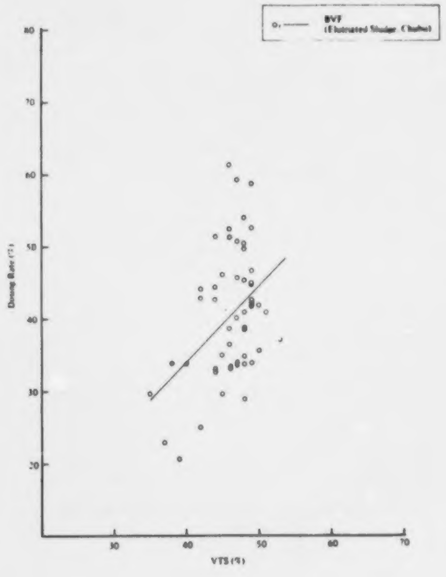


Fig. 1.19 Dosing Rate (Ferric Chloride) vs. VTS

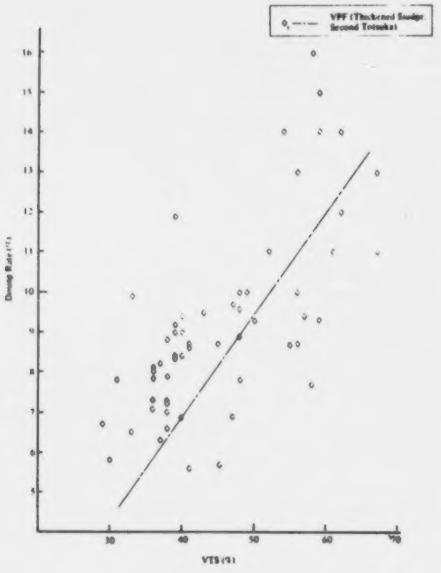


Fig. 1.20 Dosing Rate (Carbide Slurry) vs. VTS

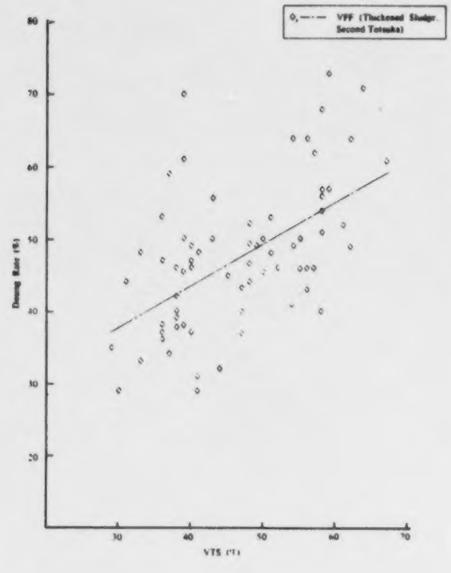


Fig. 1.21 Dosing Rate (Polymeric Coagulent) vs. VTS

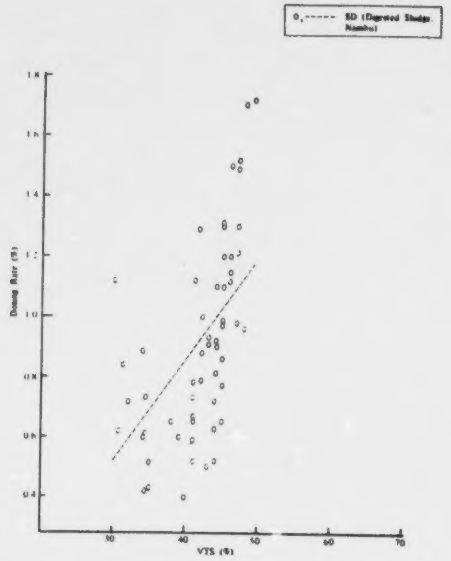


Fig. 1.22 Cake Water Content vs. VTS

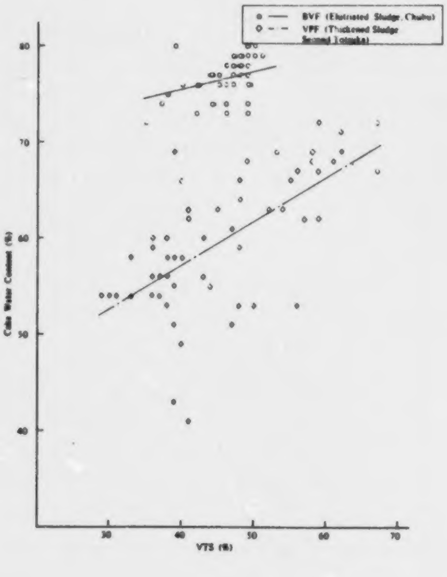
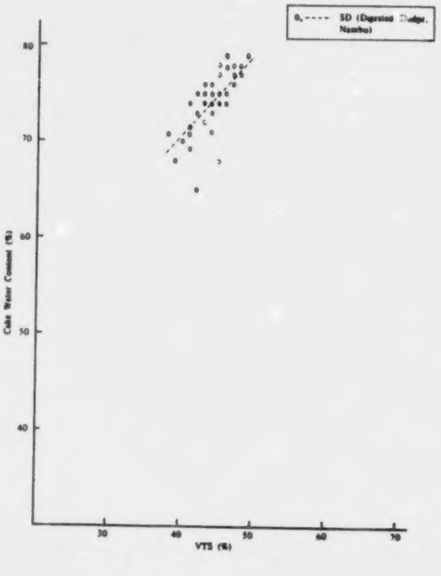


Fig. 1.23 Cake Water Content vs. VTS



1.3 EVALUATION OF THE DEWATERING MACHINES

In preceding sections the present state of the mechanical sludge-dewatering in Yokohama City and characteristics of various dewatering facilities and dewatering processes have been explained. In the following we attempt to evaluate dewatering machines on the basis of those informations.

1.3.1 THE RANGE OF COMPARISON

Dewatering efficiency, sludge management cost and the [illegible] were calculated for each of the cases listed in Table 1.9 for respective dewatering machines for comparison.

Table 1.9 Conditions for Comparison of the Dewatering Machines

Dewatering Machines			Feed Sludge		
Types	Working Time (hours)	Capacity	Kinds	VTS (%)*	Dry Solids (t/day)
BVF	7	Of Maximum Capacity Possible Now	Thickened Sludge	35 47 60	10 20 30 40 50
VPF					
HPF					
SD	24				

* VTS at 35% was the average concentration when the operation was started.
VTS at 47% was the annual average concentration in 1975.
VTS at 60% was the average concentration during winter in 1975 - 6.

Although not all of these cases are feasible, but the factors were calculated to know the main trend.

1.3.2 THE METHOD OF DETERMINING THE CAPACITY OF THE DEWATERING MACHINES

For evaluation it is important to know how the capacity of various dewatering machines changes depending upon the fluctuation of various conditions.

Here the linear functions obtained from the information in Section 1.2 in which the various properties of the machines including the dewatering capacity were studied are used. These functions are shown in Table 1.10.

The accuracy of these functions has to be determined by further research and analysis, but the main trends are to a large extent in agreement with the actual experience. As to the filtration rate and the water content of the cake for BVFs, the actual result of treating the elutriated sludge was used, and as to the dosing rate the actual result by VPFs was used.

The results in Table 1.10 are shown schematically in Figs. 1.24 ∿ 28.

In Fig. 1.24, the capacity of the HPF was calculated by assuming the cycle time of 30 minutes while that of the VPF is 20 minutes.

Other assumptions are listed in the bottom column of the Table of Comparison (cf. Table 1.11).

Table 1.10 Characteristics of the Capacity of Respective Dewatering Machines (The Results of Operation Depending Upon VTS)

X = VTS

Types of Machines / Items			BVF	VPF	SD
Dewatering Rate (Kg/m^2·h)			33.06−0.415X Υ=0.409**(ϕ=50)	15.3−0.187X Υ=−0.802**(ϕ=71)	1200^{*1}(Kg/h)
Dosing Rates (%)	Thickened Sludge VTS30∿60% Ave.47%	Carbide Slurry	Same as Right	20.2+0.583X Υ=0.567**(ϕ=71)	(Polymeric Coagulant) −0.5+0.034X Υ=0.528**(ϕ=60) (30 ≤ X ≤ 50)
		Ferric Chloride	Same as Right	1.0+0.188X Υ=0.667**(ϕ=71)	
Cake Water Content (%)			68.3+0.181X Υ=0.286*(ϕ=50)	38.9+0.455X Υ=0.671**(ϕ=71)	37.1+0.844X Υ=0.687**(ϕ=45)

* Significant at $P < 0.05$

** Significant at $P < 0.01$

*1 The rated capacity of the SD is assumed 50 m^3/h provided TS is 3%. The dewatering rate is assumed 80% of the rated rate after the past performance in city's treatment plants.

Fig. 1.24 Filtration Rate vs. VTS

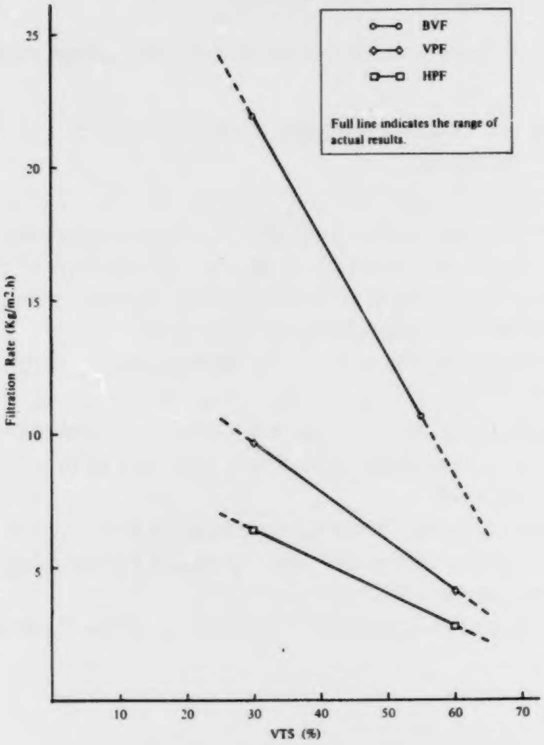


Fig. 1.25 Dosing Rate (Carbide Slurry) vs. VTS

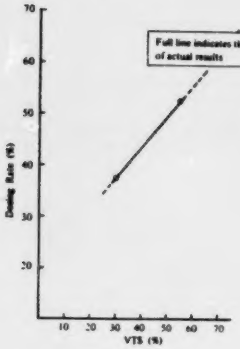


Fig. 1.26 Dosing Rate (Ferric Chloride) vs. VTS

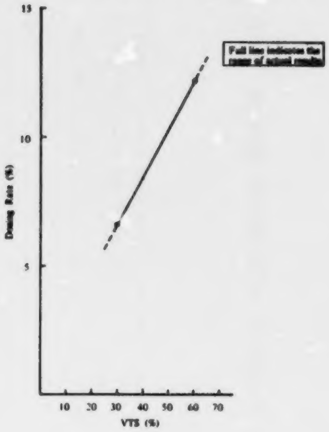


Fig. 1.27 Dosing Rate (Polymeric Coagulant) vs. VTS

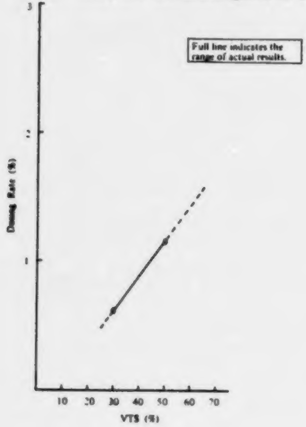


Fig. 1.28 Cake Water Content vs. VTS

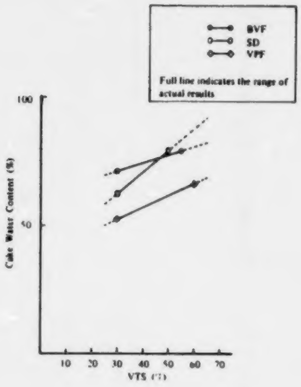


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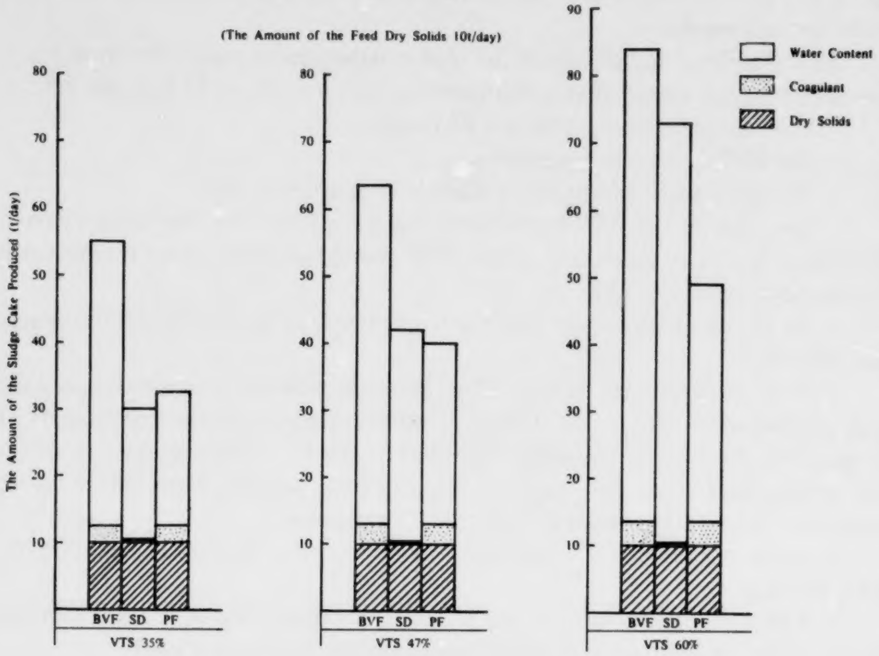
Table 1.11 Economic Comparison of Respective Dewatering Facilities (Thickened Sludge, VTS 47%)

Types of Dewatering Machines	Feed Dry Solids	Dewatering Machine		Coagulant Cost						The Amount Produced of Sludge Cake and the Disposal Cost		Electric Cost		Repair Cost ($/year)	Personnel Cost		Depreciation Cost ($/year)	Annual Sludge Management Cost ($)	Sludge Management Cost per Ton of Feed Dry Solids ($)	Installation Area (m²)	Composition of Sludge Cake	Remarks
		Specification	Number	Carbide Slurry		Ferric Chloride		Polymeric Coagulant		Amount Produced	Annual Disposal Cost ($)	Maximum Demand (KW)	Electric Energy (KWH/day)		Operators	Annual Cost ($)						
				Amount	Annual Cost	Amount	Annual Cost	Amount	Annual Cost			Standing Charge ($/year)	Energy Charge ($/year)									
BVF	10 t/day 2,950 t/year	Filtration Area 131 m² Drum Diameter φ4,200 mm Drum Width 10,000 mm Drum Rotational Frequency 0.125~0.6 rpm Filtration Rate 13.9 kg/m²·h Feeding Rate of Dry Solids 1,820 kg/h Dosing Rates Carbide Slurry 40% Ferric Chloride 10% Cake Water Content 76.5%	2	24 t/day 7,080 t/year	$ 49,300	2.7 t/day 796.5 t/year	$ 74,300			67 t/day 19,763 t/year	291,700	195 8,700	1,300 12,700	8,700	1	10,000	63,000	518,300	175.7	960	Dry Solids : Coagulant : Water Content 1 : 0.58 : 5.12	
	20 5,900		3	48 14,160	98,700	5.4 1,593	148,700			134 39,530	583,300	390 17,300	2,700 26,700	17,700	2	20,000	91,300	1,003,700	170.0	1,250		
	30 8,850		4	72 21,240	148,000	8.1 2,389.5	223,000			201 59,295	875,000	585 26,000	4,100 40,000	26,000	2	20,000	112,700	1,470,700	166.9	1,570		
	40 11,800		5	96 28,320	197,300	10.8 3,186	297,300			268 79,060	1,166,700	780 34,700	5,500 53,300	35,000	3	30,000	139,700	1,954,000	165.7	1,980		
	50 14,750		6	120 35,400	246,700	13.5 3,982.5	371,700			335 98,825	1,456,700	975 43,300	6,800 66,700	43,300	4	40,000	167,700	2,436,000	165.0	2,200		
SD	10 2,950	Flow Rate 50.9 m³/h Feeding Rate of Dry Solids 1,200 kg/h Rotational Frequency 2,500 rpm Centrifugal Effect max. 3,000 G Dosing Rate Polymeric Coagulant 1.14 Cake Water Content 76	2					0.114 t/day 33.6 t/year	$145,700	44 12,980	191,300	320 14,300	2,660 26,000	10,300	3	30,000	22,800	439,900	149.1	300	1 : 0.056 : 3.36	
	20 5,900		2					0.228 67.26	291,400	88 25,960	382,600	330 14,300	5,320 52,300	20,600	3	30,000	22,300	813,300	137.9	350		
	30 8,850		3					0.342 100.89	437,100	132 38,940	573,900	400 21,700	6,000 59,000	30,900	3	30,000	36,000	1,188,600	134.3	550		
	40 11,800		3					0.456 134.52	582,800	176 51,920	765,200	400 21,700	7,970 78,300	41,200	3	30,000	36,000	1,515,200	128.4	550		
	50 14,750		4					0.57 168.15	728,300	220 64,900	956,300	640 29,000	8,980 87,700	51,500	3	30,000	49,300	1,932,100	131.0	750		
VPF	10 2,950	Filtration Area 62.5 m² Cycle Time 20 min Filtrating Pressure 3.5 kg/cm² Squeezing Pressure 12~15 kg/cm² Drying Pressure 1.0~1.5 kg/cm² Filter Cloth Washing 10~15 kg/cm² Filtration Rate 6.5 kg/m²·h Feeding Rate of Dry Solids 400 kg/h Dosing Rates Carbide Slurry 40% Ferric Chloride 10% Cake Water Content 60%	6	24 7,080	69,300	2.7 796.5	74,300			40 11,800	174,000	460 20,700	1,980 19,700	16,000	2	20,000	104,000	478,000	162.0	1,170	1 : 0.6 : 2.4	
	20 5,900		10	48 14,160	98,700	5.4 1,593	148,000			80 23,600	348,000	790 35,700	3,790 37,300	32,300	4	40,000	197,000	957,000	162.3	1,890		
	30 8,850		15	72 21,240	148,000	8.1 2,389.5	223,000			120 35,400	522,000	1,290 58,300	6,630 67,300	47,700	6	60,000	256,300	1,382,700	156.3	2,510		
	40 11,800		19	96 28,320	197,100	10.3 3,186	297,300			160 47,200	696,000	1,590 71,700	7,660 75,300	64,700	7	70,000	322,000	1,796,300	152.0	3,100		
	50 14,750		24	120 35,400	246,700	13.5 3,982.5	371,700			200 59,000	870,000	2,120 95,700	10,000 99,000	81,700	9	90,000	407,000	2,266,000	153.7	3,710		
HPF	10 2,950	Filtration Area 200 m² Cycle Time 80 min Filtrating Pressure 3~5 kg/cm² Squeezing Pressure 12~15 kg/cm² Drying Pressure 0.5~1.5 kg/cm² Filter Cloth Washing 10~15 kg/cm² Filtration Rate 4.5 kg/m²·h Feeding Rate of Dry Solids 860 kg/h Dosing Rates Carbide Slurry 40% Ferric Chloride 10% Cake Water Content 60%	3	24 7,080	69,300	2.7 796.5	74,300			40 11,800	174,000	260 11,700	1,900 18,700	11,300	1	10,000	62,700	412,000	139.7	980	1 : 0.6 : 2.4	
	20 5,900		5	48 14,160	98,700	5.4 1,593	148,700			80 23,600	348,000	475 21,300	3,470 34,000	22,700	2	20,000	136,700	830,000	140.7	1,550		
	30 8,850		7	72 21,240	148,000	8.1 2,389.5	223,000			120 35,400	522,000	700 31,700	5,110 50,300	34,000	3	30,000	185,300	1,224,300	138.3	2,175		
	40 11,800		9	96 28,320	197,300	10.8 3,186	297,300			160 47,200	696,000	1,000 45,000	7,300 71,700	45,300	4	40,000	239,300	1,632,000	138.3	2,910		
	50 14,750		11	120 35,400	246,700	13.5 3,982.5	371,700			200 59,000	870,000	1,110 65,000	8,100 79,700	56,700	4	40,000	290,300	2,020,000	136.7	3,440		
Grounds for Estimation		Days of Operation per Year 295 days Working Ratio (Same for all of the Dewatering Machines) 0.8 Operating time per unit is 7 hours a day But that of SD is 24 hours Regression formula of cake water content from VPF is applied to HPF		Unit Cost of Coagulants Carbide Slurry $6.9/t Ferric Chloride $93.3/t Polymeric Coagulant $4,330/t						Unit Cost of Disposal $14.75/t		Unit Standing Charge $3.76/KW Month Unit Energy Charge $33.3/10³KWH		BVF $4.3/[illegible] SD $4.3/[illegible] VPF $1.8/[illegible] HPF $2.6/[illegible]	Operators per Unit BVF 0.8 SD 0.2 PF 0.4		By Fixed Amount Method Life Time (years) Mechanical Equipment 20 Architecture 40					

Fig. 1.29 The Amount and the Composition of the Sludge Cake Produced (Thickened Sludge) Depending Upon VTS



1.3.3 EVALUATION

An example of the results of calculation within the aforementioned range and conditions is shown in Table 1.11 (thickened sludge, VTS 47%).

a. The amount and the composition of the sludge cake produced

The amount and the composition of the sludge cake produced by the dewatering process have much influence upon the method and cost of the succeeding treatment and disposal of the sludge. An example of the change in the amount and the composition of the sludge cake produced is shown in Fig. 1.29 (thickened sludge).

– The amount of the cake produced is determined by the water content of the cake and the amount of the coagulant used. The water content of the cake is higher at a higher level of VTS, and its influence is most evident in SDs (cf. Fig. 1.28).

– BVFs produce the largest amount of the cake. This is mainly because of the high water content of the cake, and, in comparison with SDs, is due to the greater amount of the coagulant used.

– PFs can maintain a low level of water content and hence produce less cake.

– SDs require lower dosing rate, and hence produce nearly the same amount of cake as PFs when VTS is not high.

b. Sludge management cost

The change in the dewatering cost per ton of the feed dry solids is shown in Fig. 1.30. The treatment cost was averaged over various amounts of the feed dry solids for each machine.

An example of the details of the sludge management cost is shown in Fig. 1.31 (thickened sludge, annual sludge management cost at a rate of 50 tons per day).

– The cost increases rapidly as VTS increases.

– The BVF is the most expensive.

– The SD is the least expensive when VTS is not very high.

– The cost of the HPF is relatively low mainly because the water content of the cake is low in comparison with the BVF and the capacity of the machine is large in comparison with the VPF.

– As to the detail of the cost, the disposal cost is the largest and the coagulant cost follows it.

– It is characteristic to large cities in Japan that the disposal cost occupies a large portion of the total cost. Hence, if we exclude it from the total cost, the cost of the BVF and the SD is smaller than that of the PF. However, the cost of drying and incineration is greater than the disposal cost. Therefore the machines which produce cakes with low water content are advantageous.

– As to the fixed cost other than the disposal cost and the coagulant cost, the SD is the cheapest.

– The depreciation cost of the SD is the smallest. This is because SDs require small number of auxiliary devices and hence the building cost is small.

– There are many kinds of polymeric coagulants used for the SD. The function for the dosing rate used for the present analysis holds for only one kind of the coagulant, and the dosing rate is assumed to increase in proportion to VTS. Therefore, if other appropriate coagulants are used when VTS is high, the cost may become considerably low.

Fig. 1.30 Sludge Management Cost per Ton of Feed Dry Solids (Thickened Sludge) Depending Upon VTS

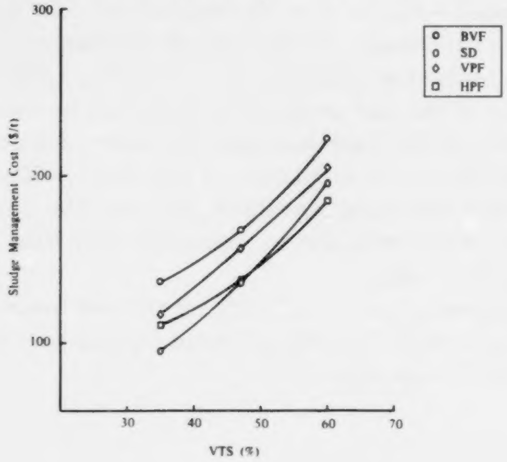


Fig. 1.31 Annual Sludge Management Cost Depending Upon VTS

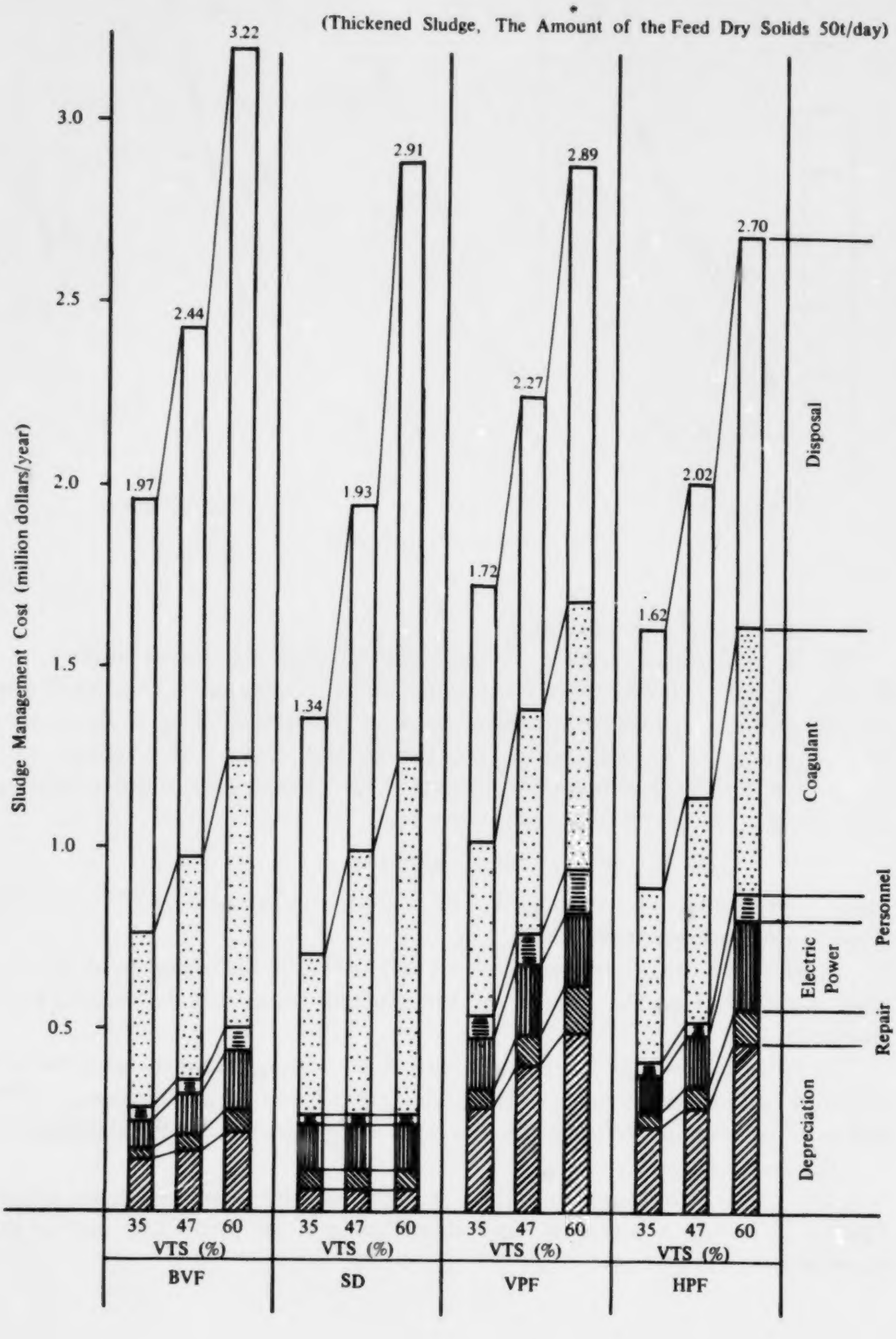
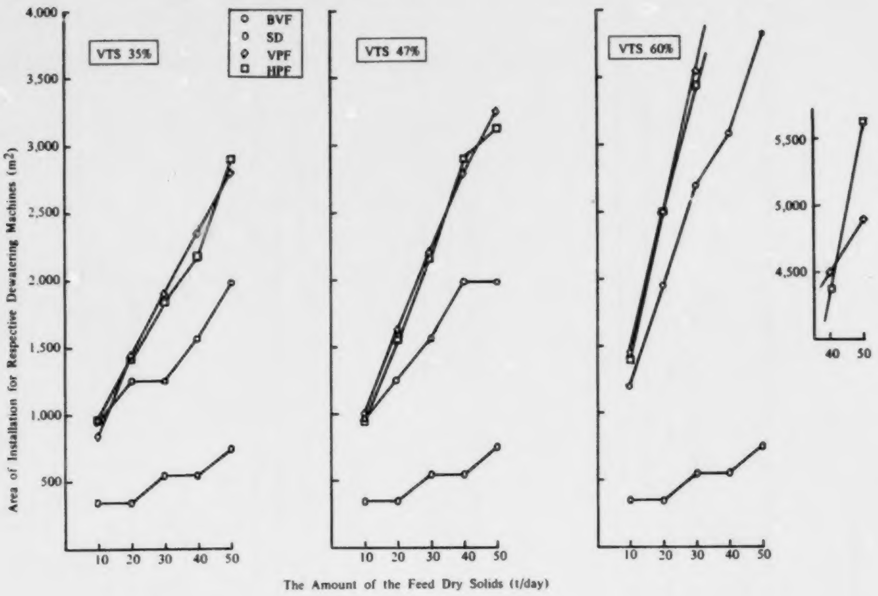


Fig. 1.32 Area of Installation for Respective Dewatering Machines Depending Upon VTS



c. The area of the installation

In Japan the land available for the treatment plant is generally limited due to the shortage of land. The space for installation is also a big problem in modification and expansion of the sludge treatment facilities. Therefore, in Fig. 1.32 are shown areas of installation per daily feeding rate for respective dewatering machines.

– The area of installation is generally large for machines which require auxiliary devices. The area is large in the order of

a PF, a BVF and an SD.

There is little difference between a VPF and an HPF because an HPF has a large capacity per a machine than a VPF.

– The performance of the SD is not influenced by VTS because of its mechanism of dewatering. The number of the machine required is determined by the feeding rate.

– In the PF and the BVF, the increase in VTS brings about the decrease in the filtration rate and the increase in the dosing rate. Therefore, an increase in VTS as well as in the feeding rate brings about a rapid increase in the area of installation.

d. Maintenance

The characteristics and problems in operation and maintenance are listed in Table 1.2. Though it is difficult to evaluate, some of the points that may be mentioned are as follows.

– The operation of the dewatering process is superior in the order of

an SD, a PF and a BVF

taking continuous automatic operation as the standard.

– As to the treatment process of the sludge cake (drying and incineration), the machines are advantageous in the order of

a PF, an SD and a BVF

from the view point of the water content of the cake.

– Disposal of the sludge cake (land fill) is easy in the order of

a PF, a BVF and an SD

because of the properties of the sludge.

e. Some comments on the evaluation

Evaluation of various dewatering machines has been attempted on the basis of the operating experience. Some points which should be kept in mind will be listed.

– The data for determination of the capacity of the dewatering machines were obtained by analysis of the actual operation, and hence do not necessarily reflect the true causal relation. Therefore, some of the actual values may change drastically by further research and analysis.

– As to the annual sludge management cost, because both the quality and the quantity of the sludge fluctuate, the cost should be integrated for each dewatering machine according to this fluctuation to know the cost for respective treatment plants.

– The sludge management cost is largely influenced by such factors as the length of the operating time or the capacity of a machine rather than the type of the machines. Therefore, the estimation of the amount of the sludge produced, determination of the facilities and the building program based upon the estimate are important as well as the selection of the machine.

– It is expected that the dewatering efficiency of the same type of machine may be different depending upon the total sludge treatment system in which the machine is incorporated.

The ultimate step of the evaluation of the dewatering machine is *the choice of the dewatering machine.* For this purpose, it must be first determined into what kind of sludge treatment and disposal system the dewatering process should be incorporated. The dewatering machine which is most effective and minimizes the total cost within the system must be selected. For such a selection, analysis of processes other than dewatering is necessary, which will be considered elsewhere.

CHAPTER 2
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2.6 Conclusion 135

2.1 INTRODUCTION

The area of sewerage-service region (sewered area) in Osaka City has been increased to 161.80km^2 as of Dec. 20, 1976, meaning that the ratio of the sewered area to urban area has attained a figure of 88%.

All 12 sewage treatment plants planned to be constructed are in operation, though some of them are being operated by sedimentation process, and they treated sewage at an average rate of 2,315,000m^3/day in the year between April 1975 and March 1976 to produce dewatered cake of about 500 tons per day.

In Osaka City, the area of which is narrow and mostly urbanized, a site for sludge disposal is difficult to secure, so that the treatment and the ultimate disposal of sludge is a very serious problem in sewerage works. Fortunately the problem is being solved for the time being by using sludge for sea-reclamation works in the north port establishing service in Osaka Bay, but these reclamation works will be finished in 1985 according to the present schedule and after 1985 it is considered impossible to secure a new disposal site in Osaka City.

So far, some sewage sludge, incinerated after mechanical dewatering process by using such coagulants as ferric chloride, ferrous sulfate and slaked lime, has been disposed to the above-mentioned sea-reclamation area in Osaka Bay. In this dewatering process, the dosage of so much slaked lime (20 to 50% to dry solid in weight) provides consequently sludge cake in large quantity. Moreover, when the sludge cake is incinerated to reduce its quantity, it would not provide function of self-combustion as the quantity of inorganic substances is relatively large, and it requires not only a large quantity of supplementary fuel but also incineration produces a great amount of ash. Thus the purpose of the incineration can not be completely achieved.

To solve this problem, the sludge conditioning process using hydrogen peroxide was developed through the discovery of the following mechanism;. Organic substances in sludge are oxidized like a chain reaction by the strong oxidizing action of the hydroxyl radical generated from hydrogen peroxide in the presence of ferric ion and also their gel structures are destroyed to isolate water in an accelerated manner, while ferrous ion turns into ferricion with a high coagulation effect which coagulate sludge particles to form flocs.

Thus dewatering of sewage sludge can be accelelated by two actions, oxidation and coagulation.

2.2 FUNDAMENTAL STUDY

After the effects of a dosage of chemicals and stirring on the dewatering characteristic of raw and digested sludges were studied by leaf test in the laboratory, the operating condition to get design criteria was researched using a pilot-scale plant of vacuum filter.

2.2.1 LEAF TEST

Mixing Tank:	Diameter 15cm x Depth 20cm	
Mixing Equipment:	Diameter 10cm	
	Mixing velocity	0 to 1,200 rpm
	Fan turbine impeller	(6-blades)

Leaf Tester:	Area of filtering surface	19.6cm²
	Degree of vacuum	500mmHg
	Filter cloth	Shikishima canvas N74

Both raw and digested sludge collected from the Tsumori treatment plant were studied by using the above-described apparatus as follows.

a. Dosage of chemicals

A sludge sample of 2 liters was collected in the mixing tank, ferrous sulfate and hydrogen peroxide were added in varied quantities, and the mixture was stirred for 10 minutes at the stirring velocity of 200 to 300 rpm. Then the leaf test was performed to measure the rate of filtration, the water content of the cake, and the releasability.

The relationship between the filtration rate and the dosage of hydrogen peroxide at a definite dosage of ferrous sulfate is shown in Fig. 2-1 and in Fig. 2-2. And the relationship between the filtration rate and the dosage of ferrous sulfate at a definite dosage of hydrogen peroxide is shown in Fig. 2-3 and in Fig. 2-4.

Fig. 2-1 Relationship between Filtration rate and Dosage of Hydrogen Peroxide

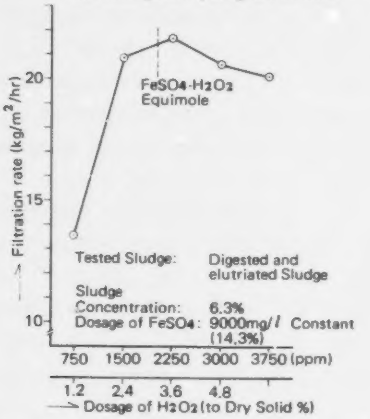


Fig. 2-2 Relationship between Filtration Rate and Dosage of Hydrogen Peroxide

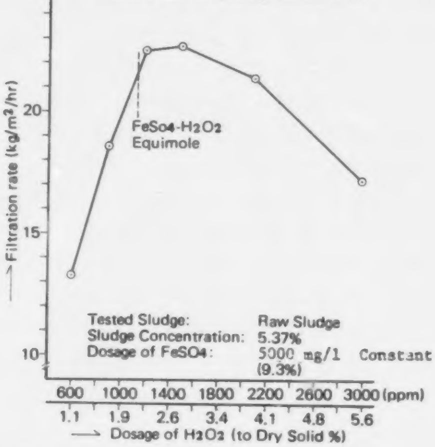


Fig. 2-3 Relationship between Filtration Rate and Dosage of Ferrous Sulfate

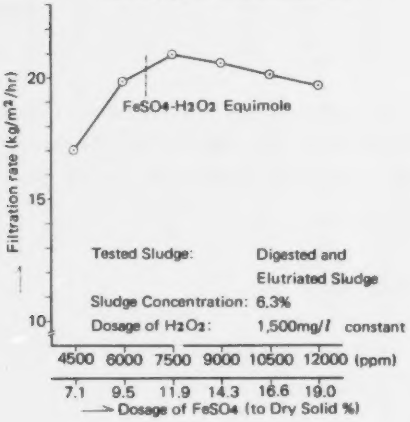
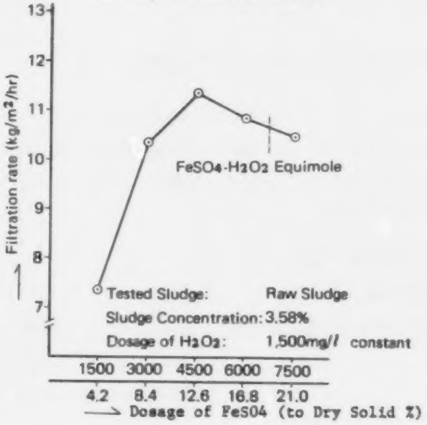
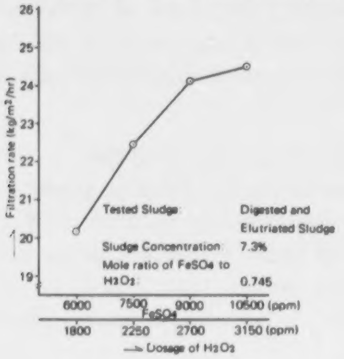


Fig. 2-4 Relationship between Filtration Rate and Dosage of Ferrous Sulfate



From Fig. 2-1, 2, 3 and 4, the filtration rate indicates approximately its maximum value near the equimolecular value of ferrous sulfate to hydrogen peroxide. Then the relationship between the filtration rate and the dosage of chemicals at a definite mole ratio of ferrous sulfate to hydrogen peroxide is shown in Fig. 2-5.

Fig. 2-5 Relationship between Filtration Rate and Dosage of Chemical at a constant mole ratio



From the above results it has become obvious that this process can be used for dewatering if ferrous sulfate of 10 to 15% to dry solid in weight is added and hydrogen peroxide of 1,500 to 2,500 ppm to sludge slurry in volume is added for raw sludge or for digested sludge, and that the dewatering process can be further improved in the region of the mole ratio of 0.6 to 1.2 of ferrous sulfate to hydrogen peroxide.

b. Flocculating condition

After a sludge sample of 2 liters was poured into the mixing tank and hydrogen peroxide at 1500 mg/l and ferrous sulfate of 6000 mg/l were added, the effects of the mixing velocity and mixing time on the fitration rate were studied.

The relationship between the filtration rate and the stirring time at a stirring velocity of 100 to 300 rpm is shown in Fig. 2-6 and that at the mixing velocity of 300 to 900 rpm is shown in Fig. 2-7.

Fig. 2-6 Effect of Stirring Velocity and Stirring time on Filtration rate.

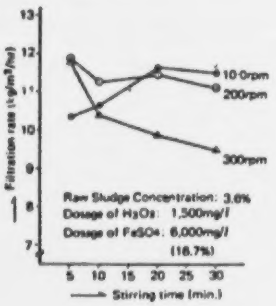
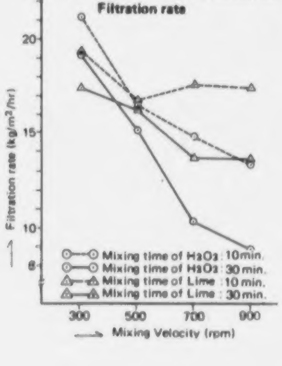


Fig. 2-7 Effect of Mixing Velocity on Filtration rate

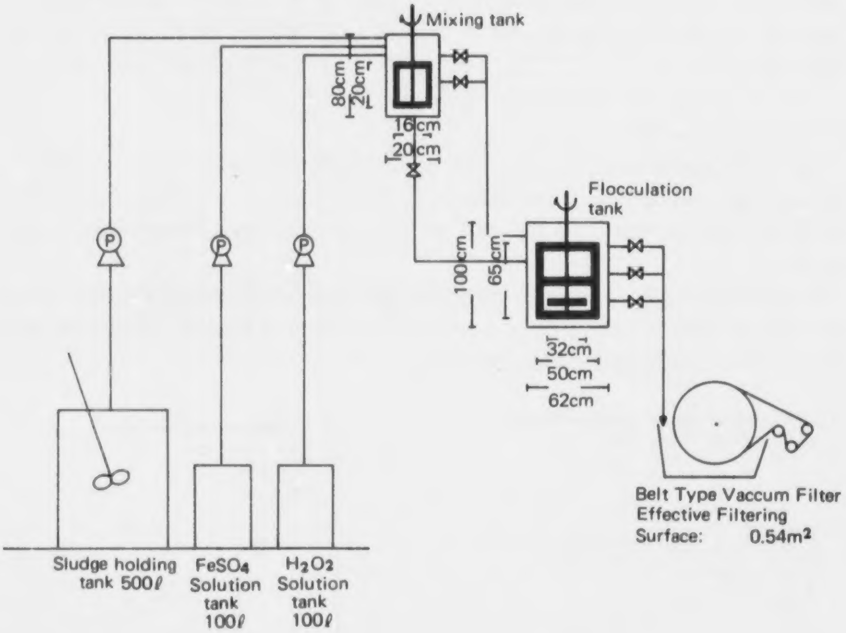


According to the above results, the mixing tank should be designed to mix its contents homogeneously in as short a time as possible, while the flocculation tank should be designed to provide as slow a velocity as possible in order not to give excess shear stress. As the sewage sludge is a Bingham fluid, of which the yield point becomes higher with higher concentration,the sludge can not be mixed homogeneously without a considerably high rotation when the diameter of an impeller to that of the tank is small, but the shear stress is consequently increased to destroy flocs at high rotation. Therefore slow rotation and a large ratio of the diameter of the impeller compared to that of the tank are advantageous for a flocculation tank.

2.2.2 PILOT PLANT TEST

A dewatering test was carried out by using the apparatus shown in Fig. 2-8 to confirm the leaf test results under nearly the same conditions as at an actual plant, and by using a sample of raw sludge mixed with digested sludge from the Tsumori treatment plant. The effects of added chemicals, sludge concentration, the vacuum filter operating condition, and sludge flocculation on the filtration rate were examined in checking its stability as regards continuous operation.

Fig. 2-8 Outline of Pilot Plant



From those results shown in Fig. 2-9 to Fig. 2-14, the following was confirmed.

Fig. 2-9 Relationship between Filtration Rate and Dosage of Hydrogen Peroxide

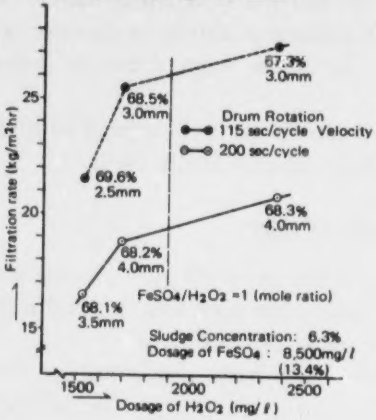


Fig. 2-10 Relationship between Filtraton rate and Sludge Concentration

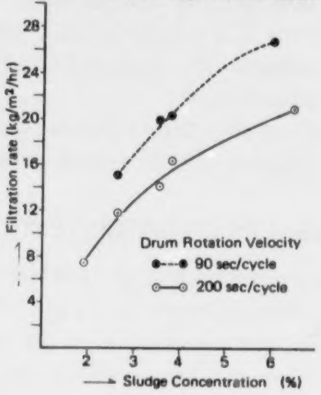


Fig. 2-11 Relationship between Filtration Rate and Drum Rotation Velocity or Submerged Filter Surface Ratio

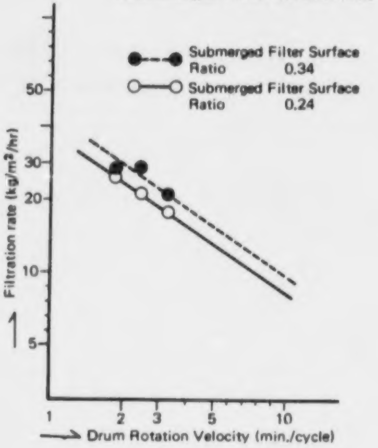


Fig. 2-12 Effect of Stirring Velocity on Filtration Rate

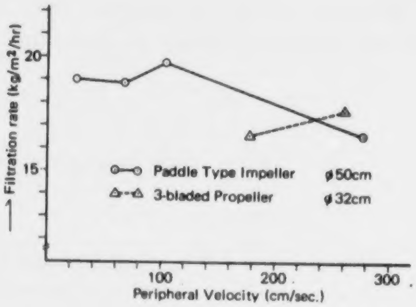


Fig. 2-13 Relationship between Filtration Rate and Stirring Velocity or Stirring Time of Floccutator

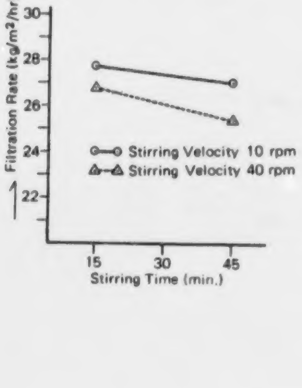
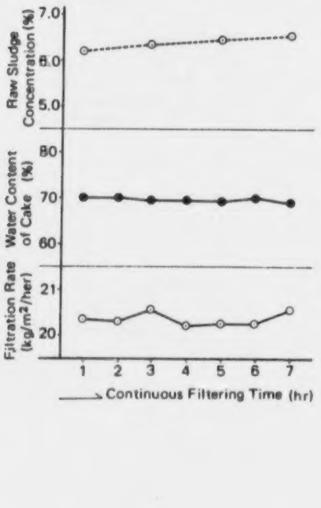


Fig. 2-14 Performance on Continuous Operation Test



- The results at this pilot plant are in close agreement with those of the leaf test. That is, sludge should be mixed homogenously in as short a time as possible in a mixing tank, while in the flocculation tank which should be used like a storage tank, sludge should be agitated as slowly as possible, an absence of sediment and an impeller having a large diameter being advantageous.
- Even the dilute slurry of sludge about 2.5% can be stably dewatered without clogging the filter cloth so that this pilot plant can be actually used.

2.3 RESULTS OF BATCH TEST IN FULL-SCALE PLANT

As the possibility of dewatering of sludge conditioned by this new process was confirmed from the results of the leaf test and the pilot scale plant test, the full-scale plant test was carried out at the Nakahama sewage treatment plant.

One of the three mixing devices, which has been previously installed, was modified for purpose of this test. The flow-sheet and the specification breakdown of this equipment are given in Fig. 2-15 and in Table 2-1 respectively.

As a sludge treatment system combined by digestion-elutriation-chemical conditioning-dewatering processes in series was being used at this treatment plant, elutriated sludge after digestion mainly was tested.

However, raw sludge was also tested to some extent taking into account of the future movement to direct incineration of raw sludge without using digestion process.

Fig. 2-15 Flow-Sheet of Full-scale Plant

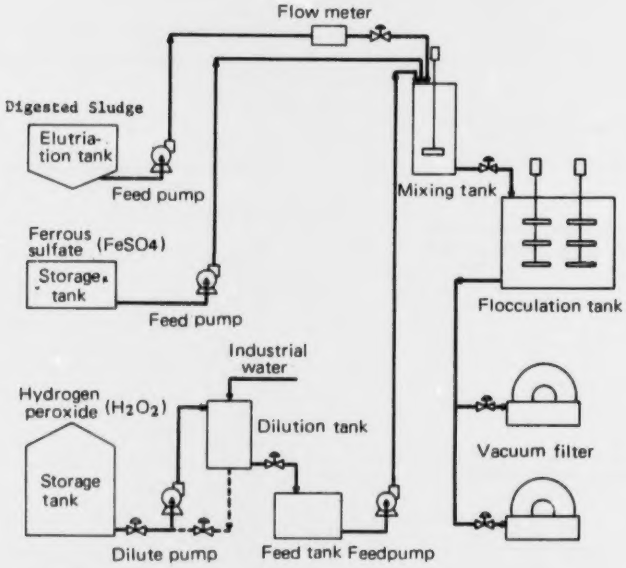


Table 2-1 Outline of Equipment

Name		Specification	
Coagulation Tank and Filter	Mixing Tank	Diameter 1.3m x Height 1.5m x max. capacity 1.7m^3 Stainless steel 304	1
	Mixer	Pitched paddle type width 0.65m x 20 ~ 85 rpm x 2.2kW	1
	Flocculation Tank	Width 1.5m x Length 3.3m x Height 1.9m x max. capacity 7m^3 Steel Inner surface coated with epoxy resin	1
	Flocculator	Pitched paddle type width 1.0m x 10 ~ 40 rpm x 3-steps impeller 0.75kw	1
	Filter	Belt type vacuum filter Filtering surface area 33m^2	2
Tested Sludge	Feed Pump	Diameter 100mm x Head 13m x capacity 1m^3/min. x 15kw	2
Ferrous Sulfate	Feed Tank	Width 2m x Length 8m x Depth 1.5m	1
	Feed Pump	Diameter 50mm x Head 13m capacity 50 *l*/min x 0.75kw	2
Hydrogen Peroxide	Storage Tank	Diameter 2.7m x Height 3.7m x Capacity 20m^3 Alminium	1
	Dilute Tank	Diameter 0.65m x Height 1.2m x Capacity 0.3m^3 Stainless steel 316	1
	Feed Tank	Horizontal type Diameter 1.1m x Length 2m x Capacity 2m^3 Alminium	1
	Dilute pump	Diameter 25mm x Head 10m x capacity 80 *l*/min. x 0.85kw Liquid-contacting part Stainless steel 316	2
	Feed Pump	Diameter 50mm x Discharge pressure 2kg/cm^2 x capacity 10 ~ 45 *l*/min. x 2.2kw Liquid-contacting part: PVC	2

The effects of the dosage of hydrogen peroxide and ferrous sulfate as well as the effects of flocculation on the filtration rate for the sludge sample listed in Table 2-2 are shown in Fig. 2-16 to Fig. 2-19. From those results, the data obtained from the leaf test and the pilot plant test were confirmed to be reproducible by this actual plant test, and sludge could obviously be dewatered up to the same level as that in the traditional process if flocculation is done carefully.

Fig. 2-16 Relationship between Filtration Rate and Dosage of Hydrogen Peroxide

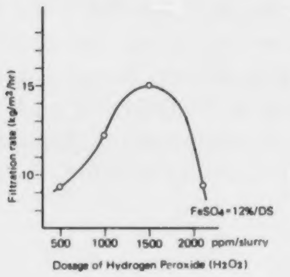


Fig. 2-17 Relationship between Filtration Rate and Dosage of Ferrous Sulfate

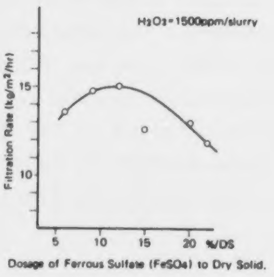


Fig. 2-18 Effect of Mixing Time on Filtration Rate

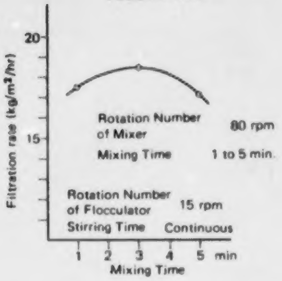


Fig. 2-19 Effect of Stirring Velocity on Filtration Rate

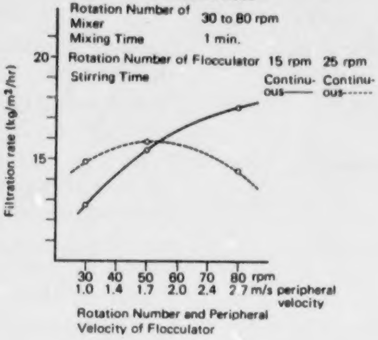


Table 2-2 Tested Sludge Properties

	pH	Alkalinity (ppm)	Dry Solid Concentration (%)	Content of Organic Matter (%)
Digested and Elutriated Sludge	7 – 8	400 – 600	3 – 4	44 – 46
Raw Sludge	6 – 7	300 – 400	3 – 4	55 – 60

Furthermore, the test using raw sludge gave the following good results when hydrogen peroxide and ferrous sulfate were added at the rates of 1,400 to 1,500 ppm to sludge in volume and at 13 to 19% to dry solid in weight respectively;

The filtration rate was 11 to 19kg/m² /hr, the water content of the sludge cake was 75 to 77% and the thickness of the dewatered sludge cake was 4mm.

2.4 RESULTS OF CONTINUOUS OPERATION AT THE FULL-SCALE PLANT

On the basis of full-scale plant test results which showed that this process can in fact be used for dewatering process, the Nakahama sewage treatment plant has been using this new process exclusively since the end of October 1976. At present 50 to 70 tons of dewatered sludge cake per day can be obtained from slurry sludge with a concentration of 3 to 4% at a rate of 400 to 500m³ /day under the following operating conditions; dosage of hydrogen peroxide – 1,200 ppm, dosage of ferrous sulfate – 15%, mixing velocity – 80 rpm, mixing time – 2.5 minutes and stirring velocity – 15 rpm.

As the sludge cake of 70 to 100 tons/day was produced by dewatering 300 to 400m³ /day of slurry sludge with the traditional process, it was considered that this new process achieved the aimed for purpose. Moreover, in the traditional process the life of the filter cloth was 300 to 400 hours at most even though it was washed with hydrochloric acid on occasion due to clogging of the filter cloth, but this new process lengthened the life to over 700 hours with no-mesh-clogging, this being due to the easy cloth-releasability of sludge cake. The life of the filter cloth will depend on the strength of the fiber making up the cloth.

2.5 COMPARISON WITH TRADITIONAL PROCESSING

Though the time the new process was actually utilized in a full scale plant was too short and the quantity of sample collected was too small to allow a complete study, this new process was compared with the traditional one and discussed using the data obtained in this fundamental research.

2.5.1 PHYSICO-CHEMICAL CHARACTERISTICS OF THE DEWATERED SLUDGE CAKE AND FILTRATE

The characteristics of the dewatered sludge cake and filtrate obtained with this new process and with the traditional one are shown in Tables 2-3 and 2-4 respectively. The dewatered sludge cake obtained with this new process contains slightly more water than that with the traditional one but sludge releasability from this new process is excellent even at a thickness of 2mm and the cake contains more organic substances, approximately the same level as that in sludge used for dewatering, than that with the traditional process. Accordingly, its heat value per unit weight is higher so that it would be better for incineration when it is made a common practice. On the other hand, pH of the filtrate indicates pH of 4 to 6 this being weakly acidic, therefore accessory apparatuses may be corroded though the corrosion depends on these used materials.

Table 2-3 Properties of Dewatered Cake

Sample	Digested and Elutriated Sludge		Raw Sludge
Measured item	H_2O_2	$Ca(OH)_2$	H_2O_2
Cake thickness (mm)	2 – 5	3 – 5	2.5 – 5
Water content of Cake (%)	72 – 80	70 – 76	72 – 80
Content of organic matter (%)	45	<40	55 – 60
Heat value (Kcal/kg)	2,500	$<2,000$	3,100

Table 2-4 Properties of Filtrate

Measured Item	H_2O_2 Method	$Ca(OH)_2$ Method
pH	4 – 6	12 – 13
SS mg/l	150 – 400	150 – 400
BOD mg/l	100 – 300	–
COD mg/l	100 – 300	–
T-Fe mg/l	20 – 1,000	10 – 20

The concentration of total iron in the filtrate from this new process sharply fluctuates from 20 to 1,000 ppm. The reason is thought to be that iron begins to exude easily when the mole ratio of ferrous sulfate to hydrogen peroxide is over 1.

In Osaka City, sludge is used for sea-reclamation at present as described above. However if the sludge is approved as a harmful industrial waste (if it does not pass the check test contained within the Harmful Industrial Waste Code established under the Cabinet Order for Implementation of the Waste Management Law) it can not be reclaimed as long as it is given no special treatment. Therefore, dewatered cakes from this new process and traditional processes and their ash incinerated with an electric furnace at 800°C in a laboratory were checked according to the established method, and this new process was compared with the traditional process. The results are shown in Table 2-5.

Table 2-5 Heavy Metal Content and Results of Exudation Test in Dewatered Sludge Cake and Incinerated Ash.

	Dewatered cake				Incinerated ash				Judgment criteria for harmful industrial wastes (for exutation)
	$Ca(OH)_2$ method		H_2O_2 method		$Ca(OH)_2$ method		H_2O_2 method		
	Dry solid	Exudation	Dry solid	Exudation	Dry solid	Exudation	Dry solid	Exudation	
Water content (%)	69.0	–	72.7	–	–	–	–	–	
Organic matter (%)	39.4	–	60.1	–	0.7	–	–	–	
pH	–	12.8	–	–	–	12.7	–	6.7	
Cd	5.48	0.01	13.0	ND	2.25	ND	24.9	ND	0.3
Pd	562	0.20	818	ND	415	0.13	1068	ND	3
T-Cr	395	ND	606	0.04	230	ND	1700	0.1	
Cr +6	–	–	–	0.09	–	0.09	–	0.09	1.5
Zn	3425	0.78	3510	3.0	2800	0.18	6870	ND	
Mn	1055	ND	563	6.1	320	ND	1420	0.6	
Ni	–	–	879	0.40	–	–	150	ND	
Cu	436	0.87	768	0.05	315	0.34	1520	ND	
As	6.7	ND	2.2	ND	3.94	ND	13.7	0.09	1.5
T-Hg	2.57	ND	3.4	0.0015	ND	ND	0.13	0.0007	0.005
T-Fe	46600	0.30	58600	43	47500	0.40	97500	0.1	
CN	35.4	ND	5.5	ND	0.37	ND	–	–	1

Heavy metal content: ppm

The content of heavy metals in the dewatered sludge cake and their ash from the traditional process in which slaked lime is added is naturally lower and the pH of the exudation from the traditional process is higher due to the addition of slaked lime so that it is difficult for heavy metals to exude.

But the exudation quantity of heavy metals in either process is lower than the regulated value.

But care should be taken here in that six valent chromium was detected in the incinerated ash from this new process. It is presumed that the six valent chromium, detected only in this case, can be attributed to the following;

- high chromium content in the incinerated ash,
- high lime content in the filtrate and high lime content in the sludge used for dewatering due to returning of filtrate and waste from the slurry vat.

At any rate the volume of dewatered sludge cake produced in this new process will be smaller than that in the traditional process, but the possibllity that three valent chromium might be oxidized (to six valent chromium) in the incineration of sludge cake can not be denied.

This problem will be further studied in full-scale plant operation in the future.

2.5.2 DISINFECTANT ACTION OF HYDROGEN PEROXIDE

In sewage sludge, various bacteria, viruses and parasites are present. The disinfectant action of hydrogen peroxide was examined by using the number of coliform groups as a contamination index for fecal coliform bacteria, which is shown in Table 2-6. The number of coliform groups measured in the sludge collected from the slurry vat of the full-scale vacuum filter is shown in Table 2-7. In each cases, the disifectant action of the hydrogen peroxide is recognized to be significant.

Table 2-6 Disinfectant Effect of Hydrogen Peroxide on No. of Coliform Group.

Sample Dosage of H_2O_2	Digested Sludge	Raw Sludge
0 (mg/l)	1.5×10^5 (No./ml)	2.4×10^6 (No./ml)
1000	3.7×10^3	4.3×10^3
1500	5.0×10^2	2.0×10^3
2000	5.0×10^1	3.0×10^1
2500	8	3

Dosage of $FeSO_4$: 6000mg/l

Table 2-7 Number of Fecal Coliform Groups in Sludge with Full-scale Plant

Sample No. Kind of sample	1	2	3
Digested and elutriated sludge	3.8×10^6	5.8×10^4	8.3×10^3
Chemical conditioning sludge	2.0×10^2	6.9×10^2	3.1×10^2

2.5.3 WORKING ENVIRONMENT

The working environment in terms of such things as dust or nasty odors often becomes a problem while sludge dewatering operation. There is no dust problem in this new process which handles no powdery lime for sludge conditioning.

For odors, 5 components, which were thought to be released from sewage sludge, of 8 substances of which the maximum contents are regulated by the Malodor Control Act were selected and their concentration measured while the filter was being operated. The results are shown in Table 2-8. From those results, this new process was considered to be effective for ammonia odor control. Though the positive deodoring effect by this new process could not be recognized only from the above results, it was significantly felt.

Table 2-8 Result of Malodoreous Substances Measurement around Filter

Sample / Measured Item	H_2O_2 method		Ca $(OH)_2$ method		Regulated standard value on site boundary
	Sludge-feeding side (inside slurry vat)	Cake-releasing side around drum	Sludge-feeding side (inside slurry vat)	Cake-releasing side around drum	
Hydrogen sulfide	ND	ND	ND	ND	20 – 200
Methyl mercaptan	ND	ND	0.3	ND	2 – 10
Methyl sulfide	1.6	0.4	1.4	0.5	10 – 200
Trimethyl amine	ND	ND	ND	ND	5 – 70
Ammonia	ND	ND	400	ND	1000 – 5000

Unit: nl/l = ppb

2.5.4 ECONOMICS

Table 2-9 shows the rough calculation of the required cost for this process in comparison with the traditional process when 100 tons of sludge as dry solid in weight per day is treated. Though some of operation and maintenace costs were difficult to estimate over a long term, it was proved that within the limits indicated in Table 2-9, this process was feasible from an economic standpoint as well.

Table 2-9 Comparison of Dewatering Process by Using Hydrogen Peroxide System with Slaked Lime System

		Chemical Conditioning Method	H_2O_2 + $FeSO_4$		$Ca(OH)_2$ + $FeSO_4$
Item		Kind of Sludge	Digested Sludge	Raw Sludge	Digested Sludge
Raw Sludge (dry solid)			100 t/day	100 t/day	100 t/day
Digested Sludge (dry solid)			70 t/day		70 t/day
Chemical Dosing Rate			H_2O_2: 2.5 – 5% (average 3.75%) $FeSO_4$: 10 – 20% (average 15%)	H_2O_2: 2.5 – 5% (average 3.75%) $FeSO_4$: 10 – 20% (average 15%)	$Ca(OH)_2$ (average 25%) $FeSO_4$ (average 5%)
Sludge Cake		Water Content	70 – 80% (average 75%)	75 – 80% (average 77%)	65 – 75% (average 72%)
		Heat Value	average 2143 Kcal/kg	average 3000 Kcal/kg	average 1714 Kcal/kg
		Products	280 t/day	435 t/day	313 t/day
Incinerated Ash Products			52 t/day	52 t/day	78.4 t/day
Cost Comparison	Construction Costs		12,944 million yen	8,203 million yen	13,798 million yen
	Operation and Maintenance Cost Per Year	Depreciation & Interest	600 million yen	410 million yen	645 million yen
		Chemical Cost	H_2O_2: 70 t/day x 0.0375 x $\frac{1}{0.35}$ x 365 x 10,000yen /t = 273,750,000 yen $FeSO_4$: 70 x 0.15 x $\frac{1}{0.5}$ x 365 x 800 = 6,130,000 yen	H_2O_2 100 t/day x 0.0375 x $\frac{1}{0.35}$ x 365 x 10,000yen/ t - 391,100,000 yen $FeSO_4$: 100 x 0.15 x $\frac{1}{0.5}$ x 365 x 800 = 8,760,000 yen	$Ca(OH)_2$: 70 t/day x 0.25 x 365 x 12,500yen/t = 79,840,000 yen $FeSO_4$: 70 x 0.05 x $\frac{1}{0.5}$ x 365 x 800 = 2,040,000 yen
			Total = 279,880,000 yen	Total = 399,860,000 yen	Total = 81,880,000 yen
		Supplemental Fuel Cost	36.65 l/t x 100 t/day x 365 x 33.80 yen/*l* = 45,220,000 yen		36.65 l/t x 100 t/day x 365 x 33.80 yen/t = 45,220,000 yen
		Boiler for Digester Heating	110.55 x 100 x 365 x 33.80 = 136,390,000 yen	69.08 l/t x 100 t/day x 365 x 33.80 yen/*l* = 85,220,000 yen	130.97 x 100 x 365 x 33.80 = 161,580,000 yen
		Furnace for Incinerator	Total = 181,610,000 yen	Total = 85,220,000 yen	Total = 206,800,000 yen
		Ultimete Disposal Costs of Ash	52 t/day x 365 x 4,922 yen/t - 93,420,000 yen	52 t/day x 365 x 4,922 yen/t = 93,420,000 yean	74.8 t/day x 365 x 4,922 yen/t = 134,380,000 yen
		Electricity Water Supply Costs	¥ 318,350,000	¥ 178,380,000	¥ 318,230,000
		Repair Material	¥ 83,000,000	¥ 63,000,000	¥ 90,000,000
		Labor Cost	¥ 400,000,000	¥ 305,000,000	¥ 400,000,000
		Total	¥ 1,956,260,000	¥ 1,534,800,000	¥ 1,876,290,000
	Operation and Maintenance Cost Per 1 Ton of Dry Solid		¥ 53,600	¥ 42,100	¥ 51,400

2.6 CONCLUSION

The merits and the demerits of this new process are summarized as follows along with the problems to be solved in the future.

Merits:

1. Smaller quantity of dewatered sludge cake and incinerated ash.
2. Higher heat value per dry solid base.
3. Simpler equipment, easier operation and maintenance.
4. No need to adjust the conditions for the addition of chemical with a slight variation in sludge properties and sludge concentration.
5. Higher sludge recovery rate due to the easy releasability of dewatered cake, longer life of filter cloth, and no need of washing filter cloth with acid solution.
6. Better working environment.

Demerits and problems:

1. Slightly higher water content in dewatered sludge cake than that in the traditional process.
2. Necessary to prevent the apparatuses from coming into contact with the liquid due to the possibility of corrosion on account of the weak acidity of sludge.
3. Slightly higher treatment cost than that with the traditional process.
4. Slight concern for the stable supply of hydrogen peroxide the manufacturers of which being oligopolistic.

The method and engineering for treating sludge should be comprehensively discussed as regards ultimate disposal of sludge and effective utilization of the sludge. In the future, this new process will be studied further in terms of systematizing the treatment and disposing of the sludge. From this point on, together with the disposal of waste gas at incineration process, the prevention of apparatus corrosion, and the structure of the incinerator to be used, the quantitative analysis of the added chemical and the mixing conditions that improve the process economically will be made the subjects of study.

CHAPTER 3. SURVEY OF ECONOMICAL AND TECHNICAL PERFORMANCE FOR EMISSION CONTROL EQUIPMENT INSTALLED WITH SLUDGE INCINERATOR

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3. SURVEY OF ECONOMICAL AND TECHNICAL PERFORMANCE FOR EMISSION CONTROL EQUIPMENT INSTALLED WITH SLUDGE INCINERATOR

3.1 INTRODUCTION

At the U.S-Japan 4th conference on Sewage Treatment Technology, a brief report was presented on the treatment and disposal of sewage sludge in Japan. In this report, it is noted the remarkable progress of construction of sewage facilities in Japan accompanied by a very rapid increase in the amount of sewage sludge which is to be disposed. See Table 3.1.

Table 3.1 Annual Change in Sludge

	Total (x10^4 persons)	ⓐ Sewered Population (x10^4 persons)	No. of S.T.P.	ⓑ Treated Sewage Volume (M·m^3/yr)	ⓒ Sludge (99% moisture) (M·m^3/yr)	ⓒ/ⓑ (%)	ⓑ/ⓐ (m^3/cap/day)
1967	10,024	1,112	142	2,371	22,93	0.97	0.58
1968	10,141	1,283	175	2,691	28,19	1.05	0.57
1973	10,871	2,110	253	5,733	104,12	1.82	0.74

Sewage sludge in 1953 and sewage volume are listed in Table 3.2.

Table 3.2 Generated Sludge Volume by Treatment Process

(1973)

Process	No. of S.T.P.	Ⓐ Sewage Volume (x10^3m^3/yr)	Ⓑ Sludge Generated (x10^3m^3/yr)	Ⓒ Night Soil (x10^3m^3/yr)	Ⓑ+Ⓒ (x10^3m^3/yr)	Ⓓ Sludge Volume (99% moisture) (x10^3m^3/yr)	Ⓔ Night Soil (99% moisture) (x10^3m^3/yr)	Ⓓ+Ⓔ (x10^3m^3/yr)	ⒷⒶ (%)	ⒹⒶ (%)
Activated Sludge process	166	3,760,401	64,770.3	2,688.1	67,458.4	86,567.8	6,720.3	93,288.1	1.72	2.30
Trickling Filter Process etc.	69	1,559,601	12,589.5	866.4	13,455.9	12,916.7	2,166.0	15,082.7	0.81	0.83
Primary Treatment Process etc.	18	412,731	2,861.4	201.7	3,063.1	4,630.5	504.3	5,134.8	0.69	1.12
Total	253	5,732,733	80,221.2	3,756.2	83,977.4	104,115.0	9,390.6	113,505.6	1.40	1.82

Approximately 66% of total treated sewage are treated by secondary treatment, activated sludge process, 27% by modified or trickling filter process and 7% by primary treatment and 81% of the sewage sludge is generated by secondary treatment process.

So the ratio of the amount of sewage sludge generated to the amount of treated sewage is 1.72% in secondary treatment, 0.81% in intermediate and 0.69% in primary treatment.

That is, the secondary treatment process generate greater amounts of sewage sludge which contain relatively higher organic content.

Table 3.3 shows what ways are selected to dispose of sewage sludge.

Table 3.3 Sludge Disposal

(1973)

Process	Sludge Volume ($10^3 m^3/yr$)	Share (%)
Reclamation (Land, Sea)	1,108.5	64.8
Disposal on Site in S.T.P. at the Sewage Treatment Plant	158.9	9.3
Dumping into Ocean	175.2	10.2
Soil Conditioning to Farm Land	158.0	9.2
Others	110.2	6.5
Total	1,710.8	100.0

Land-filling is the most common practice of sewage sludge disposal, with ocean dumping and agricultural land application both accounting for about 10% each. However, at the present, ocean dumping has been only practiced at Kitakyushu, which share very small.

In March 1976, JSWA conducted a survey on sewage sludge incineration facilities in Japan are summarized in Table 3.4.

About 25% of sewage sludge, and about half of the dewatered sewage sludge are incinerated. Most of these incinerators were located in the 3 major metropolitan areas (see Table 3.5).

This is the season why it is very difficult to find the site to dispose sewage sludge and efficiency of transportation is very low by heavy traffic conjestion in metropolitan areas.

Characteristics of sewage sludge and the performance of sludge treatment during 1975 in Tokyo, has been summarized in Table 3.6. 44% of the sewage sludge generated in Tokyo is incinerated, and then disposed to land reclamation site in the bay. Since sewer system in Tokyo is employed combined system, the contents of organic matter in the sludge is as low as 40%. In another words inert matters with surface water are induced to sludge.

The incineration of sewage sludge consumes a large amount of energy and as will be described later, regulation on atmospheric emissions are becoming more and more stringent, it is very important to study about performance of actual emission control equipment of sewage sludge incinerator. In 1975 Research and Technology Development Division of JSWA embarked on a program of research and development on the treatment and disposal of sewage sludge. This program includes evaluation of the land application of sewage sludge as soil conditioner, pyrolysis, and incinerators itself etc. This paper is an interim report on this subject and this project will be completed in 1978.

Table 3.4 Inventory of Incineration in 1975

(March 1976)

Prefecture	City	Name of S.T.P.	Sewer System	Treatment Process	Types of Sludge	Types of Dewatering	Multi-hearth Furnace	Fluid-bed Incinerator	Rotary Drying Incineration	The Rest	Dewatering Cake (t/year)	Moisture Content Average	Incinerated Sludge (Wet Cake, t/year)	Auxiliary Fuel (kℓ/year)	Digested Gas (m³/year)	Remarks
Hokkaido	Sapporo	Soseigawa	Combine	Activated Sludge	Raw	Va	17t/Dx3				40,483	76.4	2,471	(B) 145		(Suspension of Operation)
	Sapporo	Toyohiragawa	Combine Separate	Activated Sludge	Raw	Fp				30t/Dx1	14,301	50.5	10,972	(B)*1,256		*Heat Treatment
	Sapporo	Shinkawa	Combine Separate	Activated Sludge	Raw	Fp				42.5t/Dx2	16,910	44.1	16,230	(B)*1,369		*Heat Treatment
Tochigi	Utsunomiya	Tagawa	Combine	Activated Sludge	Digested	Va			30t/Dx1		4,104	78.1	3,197	263		
Saitama	Kawaguchi	Ryoke	Combine	Agro Accelerater	Raw Digested	Va	35t/Dx1				5,036	70.5	5,345	251		
	Arakawa	Arakawasagan	Combine	Activated Sludge	Raw	Fp	40t/Dx1				8,080	63.0	6,392	337		
Chiba	Chiba	Chuo	Combine Separate	Activated Sludge	Digested	Va	50t/Dx1				10,488	No Data				(Suspension of Operation)
	Ichikawa	Ichikawa Shumatsu	Combine	Activated Sludge	Raw	Va	20t/Dx1				3,832	75.0	1,032	161		
	Matsudo	Koganehara	Separate	Activated Sludge	Raw	Va	10t/Dx1				No Data	No Data				(Suspension of Operation)
	Narashino	Sodegaura	Separate	Activated Sludge	Raw	Va	5t/Dx1									(Suspension of Operation)
Tokyo	Tokyo	Sunamachi	Combine	Activated Sludge	Raw Digested	Va	150t/Dx1				498,770	Raw 79.4 Digested 79.1	50,450	1,712		
	..	..	..	..	..	..	150t/Dx1					..	42,812	1,382		
	..	..	..	..	..	..	200t/Dx1					..	67,240	2,347		
	..	..	..	..	..	..	250t/Dx1					..	77,331	1,429	1,412,420	
	..	..	..	..	..	..	300t/Dx1					..	90,765	1,616	3,434,250	
	..	Odai	Combine	Activated Sludge	Raw Digested	Va	100t/Dx1				179,145	..	16,003	345	1,401,350	
	..	..	..	..	..	..	150t/Dx1					..	33,363	1,524	7,430	
	..	..	..	..	..	..	180t/Dx1					..	51,006	980	1,270,810	
	..	..	..	..	..	..	180t/Dx1					..	57,408	573	2,504,120	
	..	Shingashi	Combine	Activated Sludge	Raw	Va	200t/Dx1				57,985	20.8	50,370	2,985		
	..	..	..	..	..	..	300t/Dx1						7,615	472		
	Hachioji	Kitano	Combine	Activated Sludge	Digested	Va	..	40t/Dx1			10,134	78.0	6,952	716	1,300	
	Machida	Tsurukawa	Separate	Activated Sludge	Raw	Cf	5t/Dx1				No Data	No Data				(Suspension of Operation)
	Tamagawa	Minamitama	Separate	Activated Sludge	Raw	Fp		20t/Dx1			2,977	63.5	2,900	241		
	Tamagawa	Kitatama	Combine	Activated Sludge	Raw	Fp		40t/Dx1			6,586	61.4	5,776	782		

Prefecture	City	Name of S.T.P.	Sewer System	Treatment Process	Types of Sludge	Types of Dewatering	Multi-hearth Furnace	Fluid-bed Incinerator	Rotary Drying Incineration	The Rest	Dewatering Cake (t/year)	Moisture Content Average	Incinerated Sludge (Wet Cake, t/year)	Auxiliary Fuel (kℓ/year)	Digested Gas (m^3/year)	Remarks
Kanagawa	Kawasaki	Iriezaki	Combine	Activated Sludge		Va			D.S. 6t/Dx5		13,350	71.0	13,350	1,150		
	Yokosuka	Shitamachi	Combine	Activated Sludge	Raw	Cf		40t/Dx1			2,994	74.9	2,994	255		
	Odawara	Kotobukicho	Separate	Activated Sludge	Raw	Va	40t/Dx1				7,131	77.0	6,708	916		
Toyama	Toyama	Joka Center	Combine Separate	High Rate Trickling Filter	Raw Digested	Va	18t/Dx1				2,733	75.0	1,307	2.7		(Suspension of Operation)
	Takaoka	Yotsuya	Combine	Activated Sludge	Raw Digested	Va	20t/Dx1				4,630	78.0	4,630	120		
	Kosugimachi	Daikakusan	Separate	Activated Sludge	Raw	Va				5.2t/Dx1	650	75.6	250	95		(Suspension of Operation)
Gifu	Gifu	Kokubu	Separate	Activated Sludge	Raw	Va		50t/Dx1			4,768	83.2	4,980	469		
	Gifu	Nanbu	Separate	Activated Sludge	Raw	Va		50t/Dx1			4,951	77.1	3,955	501		
Shizuoka	Shizuoka	Takamatsu	Combine	Activated Sludge	Raw	Va			30t/Dx2		8,284	70.0	7,212	347		
	Hamamatsu	Chubu	Combine Separate	Activated Sludge	Raw Digested	Va	30t/Dx1				5,921	75.0	2,608	521	422,980*	*Keeping Warm
Aichi	Nagoya	Yamazaki	Combine	Activated Sludge	Raw Digested	Va	150t/Dx2				107,836	74.8	59,956	2,202		
	〃	〃	〃	〃	〃	〃	40t/Dx2 80t/Dx1				No Data	No Data				(Suspension of Operation)
	Nagoya	Shibata	Combine	Activated Sludge	Raw	Va	150t/Dx2				39,240	78.1	38,722	1,657		
	Ichinomiya	Tobu	Combine	High Rate Trickling Filter	Raw	Va	5t/Dx1 10t/Dx1				4,286	70.0	632	(B) 29		
	Bisai	Bisai	Separate	Activated Sludge	Raw	Va	30t/Dx1				3,180	60.0	900	90		
Kyoto	Kyoto	Toba	Combine	Activated Sludge	Raw Digested	Va	60t/Dx1				134,330	77.3	1,779	47	140,960	
	〃	〃	〃	〃	〃	〃	60t/Dx1					77.3	4,618	310		
	〃	〃	〃	〃	〃	〃	60t/Dx1					77.3	12,659	674		
	〃	〃	〃	〃	〃	〃	150t/Dx1					〃	39,181	1,660		
	〃	〃	〃	〃	〃	〃	150t/Dx1					〃	38,303	1,632		
	〃	〃	〃	〃	〃	〃	150t/Dx1					〃	37,790	870	798,528	

Prefecture	City	Name of S.T.P.	Sewer System	Treatment Process	Types of Sludge	Types of Dewatering	Multi-hearth Furnace	Fluid-bed Incinerator	Rotary Drying Incineration	The Rest	Dewatering Cake (t/year)	Moisture Content Average	Incinerated Sludge (Wet Cake, t/year)	Auxiliary Fuel (kℓ/year)	Digested Gas (m^3/year)	Remarks
Osaka	Osaka	Tsumori	Combine	Activated Sludge	Digested	Va	100t/D×1				11,766	65.0	10,032	184	1,350,480	
	Osaka	Hoshutsu	Combine	Activated Sludge	Digested	Va	100t/D×1 200t/D×1				45,802	63.0	18,148	*1,183,176		*Gas
	Sakai	Sanpo	Combine	Activated Sludge	Raw Digested	Fp	50t/D×1 70t/D×1				21,600	68.0				(Suspension of Operation)
	Kishiwada	Isonoue	Combine	Activated Sludge		Va	15t/D×1				4,253	72.5				(Suspension of Operation)
	Ikeda	Ikeda	Combine Separate	Activated Sludge	Digested	Va	25t/D×1				4,419	80.0	4,119	327		
	Suita	Minami Suita	Combine	Activated Sludge	Raw	Fp	50t/D×1				5,641	65.0				(Suspension of Operation)
	Tondabayashi Sasayamacho	Kongo	Separate	Activated Sludge	Raw	Va		15t/D×1			3,881	No Data				(Suspension of Operation)
	Neyagawa	Konoike	Combine Separate	Activated Sludge	Raw	Va	70t/D×1				6,385 m^3/year	74.6	5,295	449		
	Neyagawa	Kawamata	Combine Separate	Activated Sludge	Raw	Fp	60t/D×1				9,033	69.0	9,033	280		
	Aigawa	Chuo	Combine Separate	Activated Sludge	Raw	Va	40t/D×1				5,091	75.5	*9,834	667		Included Takatsuki Plant
Hyogo	Kobe	Suzurandai	Separate	Activated Sludge	Raw Digested	Va	15t/D×1				1,399	20.7	468	98		
	Nishinomiya	Nishinomiya	Combine	Activated Sludge	Raw Digested	Va	30t/D×1 50t/D×1				18,052	57~77	2,730	122	285,288	
	Ashiya	Ashiya	Combine Separate	Activated Sludge	Raw	Va	27t/D×1				3,630	77.5	3,630	216		
	Akashi	Hunaue	Combine	Activated Sludge	Raw	Va	20t/D×1				3,498	78.1	3,498	546		
	Kakogawa	Ogami	Combine Separate	Activated Sludge	Raw	Fp	20t/D×1				1,651	65.0	1,141	63		
	Inagawa	Harada	Combine Separate	Activated Sludge	Raw Digested	Va	50t/D×1				28,865	75.8	24,381	3,569		
Hiroshima	Hiroshima	Enami	Combine	Activated Sludge	Raw	Va	50t/D×1				20,741	77.7	20,741	1,078		
Oita	Oita	Harakawa	Separate	Activated Sludge	Raw	Va	10t/D×1				2,400	75.0	2,400	208		
Wakayama	Wakayama	Shioya	Combine	Activated Sludge	Raw	Va	60t/D×1				11,892	78.0	10,255	No Data		
Okinawa	Okinawa Chubu	Isahama	Separate	Activated Sludge	Raw	Cf		10t/D×4			444	73.4	444	(C) 192		

Note: Raw . . . Raw Sludge Digested . . . Digested Sludge Cf . . . Centrifuge Va . . . Vacuum Filter Fp . . . Filter Press

Table 3.5

	Whole State (A)	Three Metropolitan Area (B)	B/A × 100 (%)
Incinerated Sludge ×10³W.T/yr	1,015	850	84
Number of Furnace	92	40	45

Note: Three metropolitan – Tokyo, Nagoya, Kyoto-Osaka

Table 3.6 The Status of Sludge Treatment at Metropolitan Tokyo

(in 1975)

Plants		Treated Water (m³)	Thickened Sludge (m³)	Digested Sludge (m³)	Sludge Cake (tons)	Slaked Lime (kg)	Ferric Chloride (kg)	Incinerated Cake (tons)	Remarks
Shibaura	①	353,390,500	1,097,260	1,022,380	123,168	7,057,080	3,071,010		Not Incinerate
	②	1,607,420	3,970	3,720	562	32,900	14,010		
	③	965,550	3,000	2,790	337	19,282	8,391		
Mikawashima	①	250,082,730							Transported to Sunamachi by Pipeline
	②	960,620							
	③	683,290							
Odai	①	148,154,090	2,133,470	1,144,340	179,145	10,355,900	6,943,100	179,145	
	②	591,050	10,500	4,680	772	50,660	20,120	–	
	③	404,790	5,830	*3,140	489	28,300	18,970	–	
Sunamachi	①	178,710,330	2,593,580	489,810	498,770	26,772,970	16,319,180	498,770	
	②	736,800	13,080	4,460	1,905	103,680	68,390	–	
	③	488,280	7,090	*1,430	1,363	73,150	44,588	–	
Ochiai	①	193,864,940							Transported to Odai by Pipeline
	②	808,050							
	③	529,690							
Morigasaki	①	347,319,150	1,578,760		160,424	9,145,380	3,878,570		Not Incinerate
	②	2,538,860	9,440		831	46,590	19,430		
	③	948,960	*4,960		438	24,990	10,600		
Shingashi	①	83,882,160	166,372		57,985	2,718,000	990,140	57,985	
	②	577,090	761		236	12,500	5,030	–	
	③	229,190	*467		163	*7,635	*2,781	–	
Total	①	1,555,403,900	7,569,442	2,656,530	1,019,492	56,049,420	31,202,000	735,900	
	②	7,624,340	–	–	–	–	–	–	
	③	4,249,740	–	–	–	–	–	–	

Note: ① . . . . . Yearly Total ② . . . . . Daily Max. ③ . . . . . Yearly Mean per Day * Average

Analysis of Sewage

	Raw Sewer SS (mg/ℓ)	Thickened Sludge Moisture %	Sludge Cake Moisture %	Sludge Cake Organic % (D.S.)
Mikawashima	153			
Odai	115	97.5	77.7	49.3
Sunamachi	109.5	97.4	79.1	36.3
Ochiai	123			
Shingashi	89	95.2	80.8	40.4

1975, Yearly Average

The Annual Report of the Sewerage Works Bureau of Tokyo Metropolitan Government

3.2 HISTORICAL REVIEW OF EMISSION STANDARDS

Incinerators are required to be equipped with emission control equipment in order to meet the standard set by Air Pollution Control Law. The survey conducted by the JSWA showed that installation costs of the emission control facilities accounted for as much as 30 to 50% of the incinerators itself.

Table 3.7 Chronological Regulatory Standards on Control[1)]

Law	Promulgate Date	Object Area	SOx	Particulate Matter	Harmful Matter
Smoke and Soot Regulation Act (1962-6)	1963-7	5	REC ①: 0.22% (2200 ppm) Part 0.28%	REC: 0.6 ~ 2.0 g/m^3	No
	1968-3	20	0.18% ~ 0.28%	REC: 0.6 ~ 0.6 g/m^3	No
Air Pollution Contrul Act (1968-6)	1968-12	27	RTE ②: K = 20.4 ~ 29.2	REC: 0.6 ~ 2.0 g/m^3	No
	1969-2	35	K = 11.7 ~ 26.3	REC: 0.6 ~ 2.0 g/m^3	No
	1969-7	35	US ③: K = 11.7 ~ 26.3 UES: (New Facility) K = 5.26	REC: 0.6 ~ 2.0 g/m^3	No
	1971-6	The Whole State	US: K = 11.7 ~ 26.3 UES ④: (New Facility) K = 5.26	US: 0.1 ~ 0.8 g/m^3 UES: 0.05 ~ 0.4 g/m^3	REC: Cd, 1 mg/m^3 Cl, HCl, 30 ~ 80 mg/m^3 F.1 ~ 20 mg/m^3 Pb.10~30 mg/m^3
	1972-1	The Whole State	US: K = 7.01 ~ 22.2 UES: K = 2.92 ~ 5.26	US: 0.1 ~ 0.8 g/m^3 UES: 0.05 ~ 0.4 g/m^3	REC: Cd, 1 mg/m^3 Cl, HCl, 30 ~ 80 mg/m^3 F 1 ~ 20 mg/m^3 Pb, 10~30 mg/m^3
	1973-1	The Whole State	US: K = 6.42 ~ 22.2 UES: K = 2.92 ~ 5.26	US: 0.1 ~ 0.8 g/m^3 UES: 0.05 ~ 0.4 g/m^3	REC: Cd, 1 mg/m^3 Cl, HCl, 30 ~ 80 mg/m^3 F, 1 ~ 20 mg/m^3 Pb, 10~30 mg/m^3

Note: ① REC – Regulation of Emission Concertration
② RTE – Regulation of Total Emission
③ US – Uniform Standard
④ UES – Unusual Emission Standard

Installation of Sewage sludge incinerator began in Japan around 1964, and the emission control regulations were based on the soot and dust control law 1962. The standards were not so strict ones, so exhaust gas was treated by a water spray type scrubber (cooling tower) which was sufficient to reduce soot and dust.

In 1976, legislature on Basic Law for Environmental Pollution control was enacted in 1969, followed by Air Pollution Control Law.

By 1970, the general populace had become keenly aware of pollution problems and under circumstances which increases in victims from photo-chemical smog, more stringent standard are required. So in December of the same year during the so-called "Pollution Diet", almost national laws relating to pollution control was revised. The effect of the revision to the Air Pollution Control Law was to extend the designated areas covered by the provisions of the Soot and Dust Control Act to include the whole country. All smoke generating processes in all plants throughout the country (except mining) had to meet the same emission standards designated by law. Table 3.6 show how the standards have changed within recent years. The emission control levels relevant to sewage sludge incineration are listed in Table 3.8.

Table 3.8-1 Emission Standards (Solid Waste Incinerator)*

① Pollutant	② Uniform Standard	③ Special Emission Standards	④ Progressive Emission Standards	⑤ Special Progressive Emission Standard	⑥ Local Ordinaries	⑦ Remarks
(1) Sulfur Oxdie	$q = K \times 10^{-3} He^2$ Where q: Hourly Volume of Sulfur Oxides Emitted in Units of Nm^3 H: Effective Height of Stack K: Varies According to the Region K Ranks are: 3.0, 3.5, 4.67 6.42, 8.76, 11.7 14.6, 17.5	K ranks are: 1.17 1.75 2.34	$Q = a \cdot W^b$ Q: Hourly Volume of Sulfure Oxides Emitted in Units of Nm^3 W: Fuel Conseemed (kℓ/hr) a: Value designated Prefectural Government b: 0.8 ~ 1.0 Maximum Pollutants on the Ground $Q = \frac{Cm}{Cmo} \times Qo$ Qo: Sulfur Oxides (in Nm^3/hr) Cm: Maximum Pollutant Concentration Disignated by Prefectures Government Cmo: Maximum Pollutant Concentration Depending on Qo (Vol %)	$Q = a \cdot W^b + r \cdot a (W + Wi)^b - W^b$ W, i, a, b as Same as Column ④ Wi: Aditional Fuel After Designated Date (kℓ/hr) r : 0.3 ~ 0.7 Maximum Pollutant Concentration $Q = r \frac{Cm}{Cmi} Qi$ When Additional Facilities are Equipped $Q = \frac{Cm}{Cmo + Cmi} (Qo + Qi)$ Qo, Cm, Cmo as Same as Column ④ Qi : Additional on Sulfur Oxides (in Nm^3/hr) Cmi: Max Pollutant Concentration Depending on Qi (Vol %)	No	K = 1.0, as Guideline in Some Prefectures
(2) Soot and Dust	Over 40,000 Nm^3/hr 0.2 g/Nm^3 Max. Under 40,000 Nm^3/hr 0.7 g/Nm^3	0.1 g/Nm^3 0.2 g/Nm^3	No	No	Yes	0.01 g/Nm^3 as Guideline in Some Prefectures
(3) Harmful Substances	1. Cadmium and Its Compound 1.0 mg/Nm^3 2. Chlorine 30 mg/Nm^3 3. Hydrogen Chloride 80 mg/Nm^3 4. Fluorine, Hydrogen Fluoride and Silicon Fluoride 1 ~ 20 mg/Nm^3 5. Lead and Its Compound 10 ~ 30 mg/Nm^3 6. Nitrogen Oxides 100 ~ 480 ppm	No	No	No	Yes	
(4) Specially Harmful Substances (28 Substances)	No (for the present	No (for the present)	No	No	No	
(5) Heavy Metals	No (for the present)	No (for the present)	No	No	No	

* This Standards will be Applied for the Incinerator which Capacity is Above 200 kg/hr or Grate Area is Above 2 m^2.

Table 3.8-2 Regulatory Standards on Offensive Odor Substance

① Substances	② Regulatory Standards on Boundary Lines	
① Ammonia	5 ppm	When Ammonia, Hydrogen Sulfide, Trimethyl Amine are Concern, Stack Gas Regulatory Standards are Calculated by Following Equation. $q = 0.108 \times He^2 \cdot Cm$ q: Flow Rate of Pollutant (m^3/hr) He: Ehective Height of Stack (m) Cm: Standards as Column ②
② Methyl Mercaptan	0.002 ~ 0.01	
③ Hydrogen Sulfide	0.02 ~ 0.2	
④ Methyl Sulfide	0.01 ~ 0.2	
⑤ Dimethyl Sulfide	0.009 ~ 0.1	
⑥ Trimethyl Amine	0.005 ~ 0.07	
⑦ Acet Aldhyde	0.05 ~ 0.5	
⑧ Styrene	0.4 ~ 2	

Table 3.8-3 Emission Standards of Nitrogen Oxides

Facilities		Gas Volume ($10^4 Nm^3/hr$)	Standards ppm	On (%)
1. Boilers	Gas Fud	1 ≤ < 10	130	5
		10 ≤	100	5
	Solid Fuel	1 ≤	480	6
	Others	1 ≤	150	4
2. Metal Heating Furnaces		1 ≤ < 10	150	11
		10 ≤ < 4	100	11
3. Heater for Petroleum or Other Industry		1 ≤ < 4	150	6
		4 ≤	100	6
4. Portland Cement		10 ≤	250	10
5. Nitric Acid Production		All Facilities	200	Os
6. Coke Oven		10 ≤ ~	200	7

The NO_X Emission Concentration shall be Converted through the Following Equation (Except in the Case of Nitric Acid Production Facilities).

$$c = \frac{21 - On}{21 - On} \times Cs$$

Where

C: NO_X Emission Concentration (ppm)
On: Oxygen Concentration in Stack Gas
Os: Actual Oxygen Concentration in Stack Gas
Cs: Actual Nitrogen Oxides Emission Concentration (ppm)

In 1972, scope of standard values were settled according to the Offensive Odor Control Law. Ammonium, hydrogen sulfide, and trimethylamine were stipulated according to concentrations in the air at boundary line, while methyl sulfide, trimethylamine, acetoaldehyde, and styrene were stipulated as concentration on the ground. According to the Offensive Odor Control Law, local prefecture designated the relevant areas, and determined appropriate standards. By 1977, standards had been applied in 450 cities, 463 towns, and 69 villages in 44 prefectures, plus the Tokyo metropolitan area and 10 cities designated by government ordinance. As an example, standard applied in Tokyo metropolis are shown in Table 3.9.

Table 3.9 Regulatory Standards of Offensive Odor[3)]

(Tokyo)

Land Use	Industrial Area Semi Industrial Area	Residental Area
	(ppm)	(ppm)
① Ammonia	2	1
② Methylmercapten	0.004	0.002
③ Hudrogen Sulfide	0.06	0.02
④ Methyl Sulfide	0.05	0.01
⑤ Trimethylamine	0.02	0.005
⑥ Acetaldehyde	0.1	0.05
⑦ Styrene	0.8	0.4
⑧ Methyldisulfide	0.03	0.009

Due to more stringent control of exhaust gas sewage sludge incinerators, the emission control systems has much improved.

The chronological changes in these systems have been illustrated in Fig. 3.1.

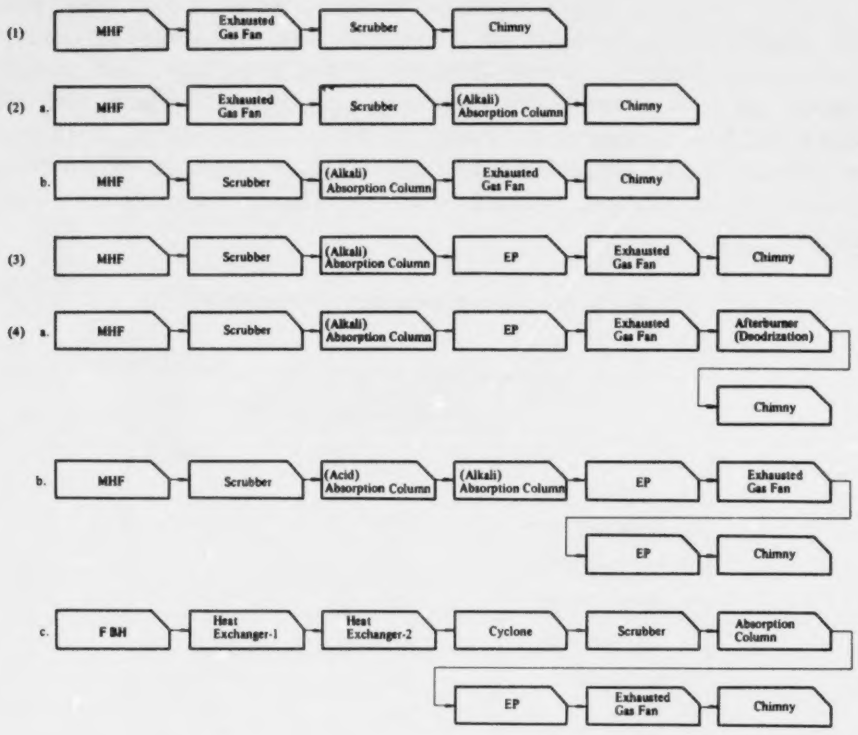
System (1), adopted from around 1964, employed washing by water in a scrubber for cooling and dust removal. It was quite sufficient at the time to reduce dust levels to the 0.7 g/Nm^3 which value designated by standard.

System (2) in 1970, adoptation of dust removal level was improved to 0.2 g/Nm^3, and because of the lower K value, for control sulfur oxides an alkali cleanser was added after the scrubber.

System (3) was adapted in 1972 due to a new local ordinances that incinerators be equipped with electrical dust precipitator (in Osaka and other some prefectures).

System (4) adopted an alkali cleansing, plus after burning for deodorery, due to the implementation of the Offensive Odor Control Law in 1972.

Fig. 3.1 Chronological Variation of Emission Control System for Sludge Incinerator



Note: MHF – Multi Hearth Furnace
EP – Electrical Dust Precipitator
FBF – Fluidized Bed Furnace
HE – Heat Exchanger

3.3 AUXILIARY FUEL AND DESIGN CAPACITY OF INCINERATOR

As of March 1975, 59 plants of the 337 presently operating public owned sewage treatment plants (that is, 17.5%) have incinerating facilities. Of the total 94 incinerators, 74% were multiple furnace, while fludized bed incinerator and rotary kilns accounted for 8 to 9%. Design capacity of each incinerator ranged from 5 tons to 300 tons of wet cake per day. All incinerators designed to handle more than 60t/day were of the multiple furnace type. Auxiliary fuels were almost always A-heavy oil, although some employed a digestion gas if it was available.

Table 3.10 lists the auxiliary fuels consumed by 36 incinerators during 1975. Average consumptions ranges were:-

standing type: 142 ~ 347 1/D.T
dynamic type: 228 ~ 561 d/D.T

This wide range in fuel consumption is due to the varying moisture content the dewatered cake, amount and type of coagulating agent, and daily operating hours.

Table 3.10 Auxiliary Fuel Consumpcion at 36 Incinerators (in 1975)

Process	Dewatering	Incineration	Design Capacity	Yearly Sludge Cake	Cake Moisture	Consumed Per Year	Fuel/ Ton-D.S.
			W.S. Tons/Day	W.S. Tons	%	Kl	ℓ/Ton-D.S.
(Thickening) ↓ (Dewatering) ↓ (Incineration)	Va	MHF	10	2,400	75.0	208	347
	Va	MHF	20	3,498	78.1	546	713
	Fp	FBF	20	2,900	63.5	241	228
	Va	MHF	27	3,630	77.5	216	264
	Va	RK	30	7,212	70.0	347	160
	Va	MHF	40	9,834	75.5	667	277
	Va	MHF	40	6,708	77.0	916	594
	Fp	FBF	40	5,776	61.4	782	351
	Cf	FBF	40	2,994	74.9	255	339
	Va	MHF	50	20,741	77.7	1,078	233
	Va	FBF	50	4,980	83.2	469	561
	Va	FBF	50	3,955	77.1	501	551
	Fp	MHF	40	6,392	63.0	337	142
	Fp	MHF	60	9,033	69.0	280	84
	Va	MHF	150	38,722	78.1	1,657	195
	Va	MHF	200	50,370	80.8	2,985	309
(Thickening) (Dewatering) (Anaerobic Digestion) ↓ (Dewatering) (Incineration)	Va	MHF	60	12,659	77.3	674	235
	Va	MHF	100	16,003	77.7	1,169	328
	Va	MHF	150	50,450	79.1	1,712	164
	Va	MHF	150	42,812	79.1	1,382	156
	Va	MHF	150	33,363	77.7	1,528	205
	Va	MHF	150	39,181	77.3	1,660	187
	Va	MHF	150	38,303	77.3	1,632	188
	Va	MHF	150	37,790	77.3	1,340	156
	Va	MHF	150	59,956	74.8	2,202	146
	Va	MHF	180	51,606	77.7	1,728	150
	Va	MHF	180	57,408	77.7	2,046	160
	Va	MHF	200	67,240	79.1	2,347	168
	Va	MHF	250	77,331	79.1	2,259	141
	Va	MHF	300	90,765	79.1	3,636	193
(Thickening) → (Anaerobic Digestion) ↓ (Incineration) ← (Dewatering)	Va	RK	30	3,197	78.1	263	376
	Va	FBF	40	6,952	78.0	716	468
	Va	MHF	100	10,032	65.0	978	279
(Thickening) → (Aerobic Digestion) ↓ (Incineration) ← (Dewatering)	Va	MHF	25	4,119	80.8	329	399
(Thickening) → (Heat Treatment) ↓ (Incineration) ← (Dewatering)	Fp	SGS	30	10,972	50.5	(B) 1,256	(B) 231
	Fp	SGS	42.5	16,230	44.1	(B) 1,369	(B) 151

Note: Va – Vacum Filtration
Fp – Filter Press
Cf – Centrifugation
MHF – Multiple Hearth Furnace
FBF – Fluidized Bed Furnace
RK – Rotary Kilne
SGS – Step Grate Storker Incinerator
(B) – Included Heat Treatment

Table 3.11 Capital Cost of Incinerator in N S.T.P.

(in 1975)

Item	
(1) Capital Costs for Civil Engg (× 10^3 ¥)	
Building Costs	80,692
Equipment Costs	603,409
Total	684,101
(2) Capital Cost of Unit Processes	
Sludge Cake Feeder	69,517
Frame of Furnace	145,986
Fuel Feeder	6,952
Air Pollution Controller	88,982
Ash Conveyer	6,952
Electric Power and Measurement	139,034
The Rest	145,986
Sub Total	603,409

Fig. 3.2 show, relationship between incinerator capacity (and hence size of incinerator) and the amount of fuel consumed. The relationship between the heat value of sewage sludge, and auxiliary fuel is now under investigation.

3.4 CASE STUDIES ON PERFORMANCE OF EMISSION CONTROL FACILITIES

In 1975, the JSWA began to investigate actual operating conditions and performance of gas emission control equipment. Survey at 13 incinerators was completed in 1975, and another 5 in 1976. The results of survey on two cases of these incinerators are described in this paper. The multiple furnace at city N P.O.S.T.P. was equipped with after burner for deodoring following the alkali washing for cleaning stack gas. The multi-stage incinerator employed in the A P.O.S.T.P of basinwide sewerage system was equipped with an electric dust precipitator.

(1) Multiple furnace at N P.O.S.T.P.

The sewage treatment plant at city N has a capacity to process 63,000 m^3/day of sewage, servicing a population of about 100,000 people. Activated sludge (step aeration) method are adopted for sewage treatment. This plant received about 18,000 m^3/day of industrial wastewater from brewery factories.

The two which from multiple furnaces are capable of handling 30 to 50 wet tons of dewatered sludge cake per day. The exhaust emission from both furnaces is integrated, and treated as shown in Fig. 3.3. The results of the survey conducted in December 1975, are shown in Fig. 3.4. Since there was no after-burner at that time, auxiliary fuel consumption amounted to 27 1/T-WS. Moisture content of the cake entering the incinerators was low, and the amounts handled were roughly 90% of design capacity. Consequently, these incinerators operated at relatively high efficienty rates, requiring no more than 342 liter of auxiliary fuel per ton of dry solid. The ratio of actual supplied air to the theoretical air requirement was 2.2, and incinerator operating temperature was at 826°C.

Construction costs have been summarized in Table 3.11. And maintenance costs etc. are listed in Tables 3.12 & 3.13.

Table 3.12 Running Costs of N S.T.P.

① Term of Operating	1,857 hr.
② Quantity of Dry Solids	2,730 ton – W.S. × (1 – 0.57) = 1,174 ton 0.632 ton – P.S./hr.
③ Personnel	¥2,910,000/person × 6 × 4/12 = ¥5,820,000
④ Fuel	Heavy Oil (A) 122 kℓ ¥33.9/ℓ × 122 × 10^3 = ¥4,136,000
⑤ Electricity	830,930 KWH ¥9/KWH × 830,930 = ¥7,478,370
⑥ Water	167,120 m^3 ¥13/m^3 × 167,120 = ¥2,172,560
⑦ Disposal of Ash	Ash 687 m^3 ¥3,330/m^3 × 687 = ¥2,287,710
⑧ Maintenance	Oil Pipe etc. ¥100,000
③ + ④ + ⑤ + ⑥ + ⑦ + ⑧	¥21,994,640

Fig. 3.2 Fuel Consumption of 33 Incinerators

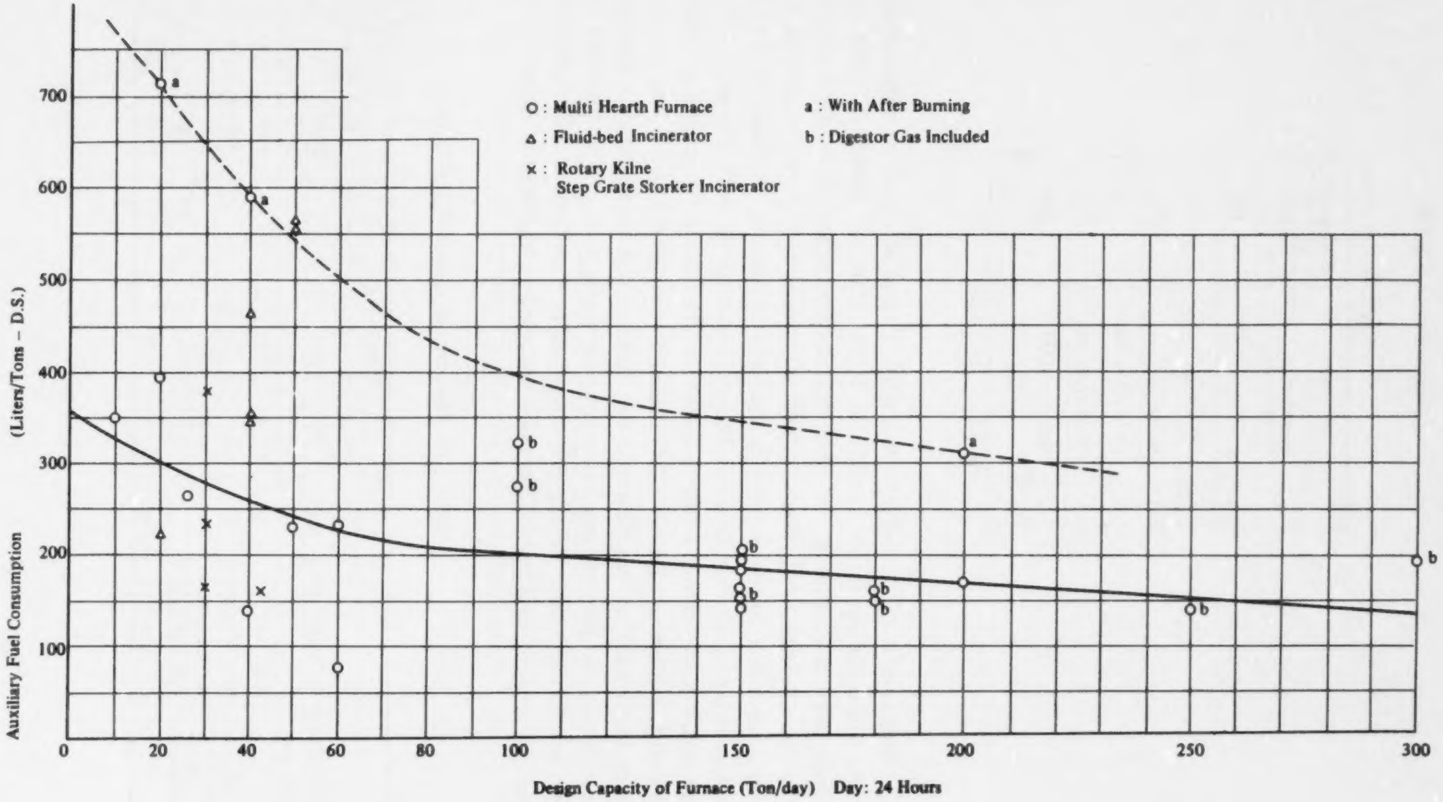


Table 3.13 Running Cost of Incinerator in N S.T.P.

(¥/Ton–D.S.) 1975

	Item	¥/Ton–D.S.
1	Personal (Attendance)	4,957
2	Fuel	3,523
3	Electricity	6,370
4	Water	1,851
5	Absorption Liquor	0
6	Disposal of Ash	1,948
7	Maintenance[1)]	2,555
	Total	21,204

Note: 1) 2% of Capital Cost.

The results of the survey conducted in August 1976 are summarized in Table 3.14, while the operating conditions at the time are shown in Table 3.15. These results show that although SO_2 and dust concentration are below the standard limits after the cooling tower, there is almost no change in NOx concentration. Odors were tested according to the 3-point comparison "odor bag" method (used by the Tokyo Metropolitan, Pollution Control Bureau), employing both physiological odor test, and analysis of offensive odor substances. In testing the degree of odor in the air around the plant, a value of 1,400 degree was measured at the washing tower outlet. Since the height of the chimney was 20 m, it was expected that dispersion would be sufficient to prevent any offensive odor being noticeable at ground level. Emission of offensive odor substances was no more than the standard values at boundary line of the plant, and hence, was not considered to be likely to cause any problem.

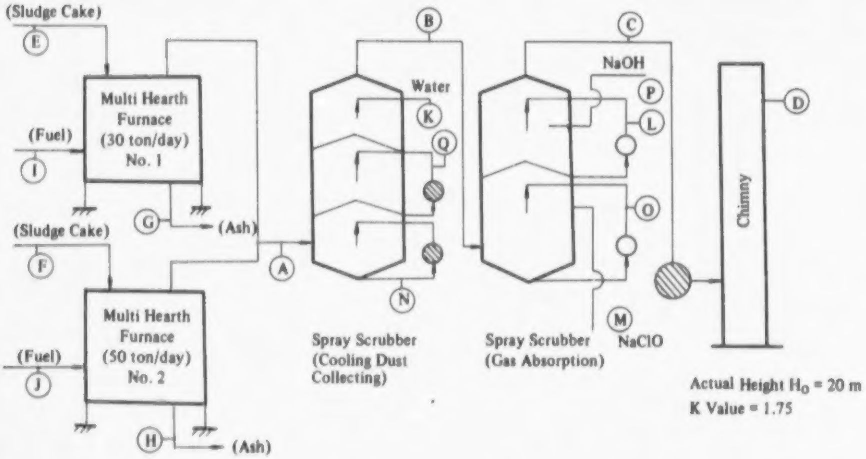
Table 3.14 Performance Data in N S.T.P.

(Aug. 20, 1976)

Item		Unit	Furnace No. 1			Furnace No. 2		
			E: Cake	G: Ash	I: A Heavy Oil	F: Cake	H: Ash	J: A Heavy Oil
Feed Cake or Discharged Ash		kg/h	1,040	136		2,000	368	
Aux. Fuel		ℓ/h			52.5			20.25
Composition Analysis of Feed Cake or Discharged Ash	Moisture	%	73.8	2.43		63.3	0.09	
	Free Moisture	%	8.4	2.6		8.4	1.4	
	Ash Content	%	55.8	89.2		53.0	94.7	
	Sulfur Content	%	2.2	No Data		1.8	No Data	
	Calorific Value	KCal/kg	1,930	600		2,150	<40	

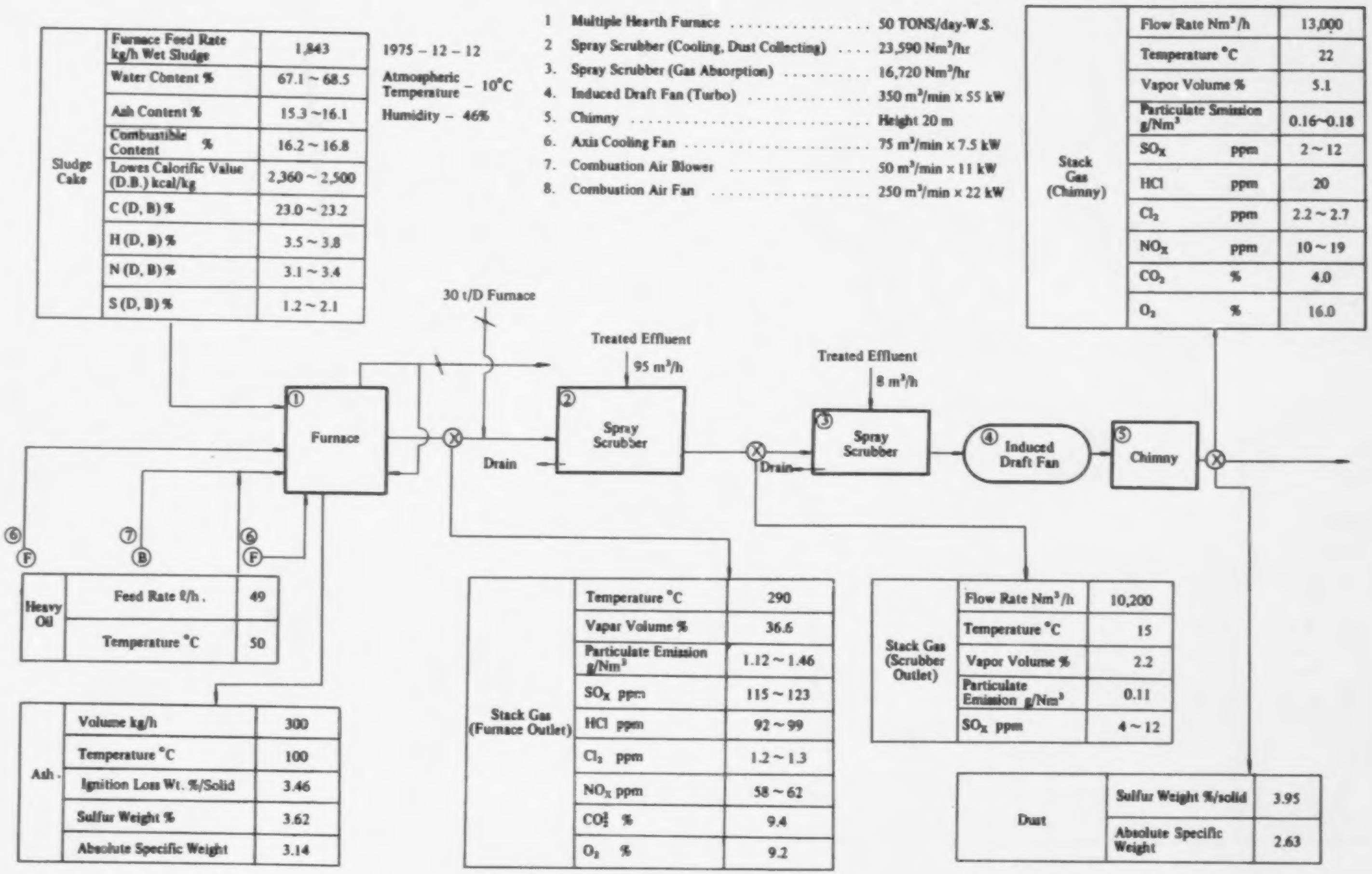
Item	Unit	A	B	C	D	Standard
Gas Temperature	°C	232	28	30	68	
Dry Gas Flow Rate	Nm^3/h	8,680	9,170	9,535	11,910	
Vapor Flow Rate	Nm^3/h	2,570	370	425	585	
Wet Gas Flow Rate	Nm^3/h	11,350	9,540	9,960	12,495	
Vapor Volume	%	23.5	3.9	4.3	4.7	
Particulate Emission	g/Nm^3	0.545	0.138	0.070	0.034	Less than 0.2
SO_2	ppm	55.8	11.1	9.8	4.3	Less than 39
NOx	ppm	99.4	85.1	81.0	71.8	
HCl	ppm	120.07	24.84	17.65	10.38	
Cl_2	ppm	1.46	1.46	6.11	2.24	
Offensive Odor	–	6,900	No Data	1,400	No Data	
Ammonia	ppm	131.54	10.80	8.15	3.83	
Trimethylamine	ppm	0.297	0.123	0.067	0.028	
Hydrogen Sulfide	ppm	1.300	0.436	0.396	0.235	
Methyl Sulfide	ppm	0.143	0.052	0.034	0.017	
Methylmercaptan	ppm	0.474	0.242	0.182	0.114	
Methyldisulfide	ppm	0.0005	0.0004	0.0002	0.0001	
Acetaldehyde	ppm	<1.0	<1.0	<1.0	<1.0	
Styrene	ppm	<0.001	<0.001	<0.001	<0.001	
O_2	%	12.1	13.0	13.5	15.8	
CO_2	%	6.7	6.2	4.3	4.2	

Fig. 3.3 Flow Diagram of Facility N


(Sludge Cake)
E
Multi Hearth Furnace (30 ton/day) No. 1
(Fuel)
I
G
(Ash)
(Sludge Cake)
F
Multi Hearth Furnace (50 ton/day) No. 2
(Fuel)
J
H
(Ash)
A
B
Water
K
Q
N
Spray Scrubber (Cooling Dust Collecting)
C
NaOH
P
L
O
M
NaClO
Spray Scrubber (Gas Absorption)
Chimney
D

Actual Height H_0 = 20 m
K Value = 1.75

Fig. 3.4 Results of Stack Gas Survey (N. S.T.P.)



1975 – 12 – 12

Atmospheric Temperature – 10°C

Humidity – 46%

	Item	Value
Sludge Cake	Furnace Feed Rate kg/h Wet Sludge	1,843
	Water Content %	67.1 ~ 68.5
	Ash Content %	15.3 ~16.1
	Combustible Content %	16.2 ~ 16.8
	Lowes Calorific Value (D.B.) kcal/kg	2,360 ~ 2,500
	C (D, B) %	23.0 ~ 23.2
	H (D, B) %	3.5 ~ 3.8
	N (D, B) %	3.1 ~ 3.4
	S (D, B) %	1.2 ~ 2.1

No.	Equipment	Capacity
1	Multiple Hearth Furnace	50 TONS/day-W.S.
2	Spray Scrubber (Cooling, Dust Collecting)	23,590 Nm³/hr
3.	Spray Scrubber (Gas Absorption)	16,720 Nm³/hr
4.	Induced Draft Fan (Turbo)	350 m³/min x 55 kW
5.	Chimny	Height 20 m
6.	Axis Cooling Fan	75 m³/min x 7.5 kW
7.	Combustion Air Blower	50 m³/min x 11 kW
8.	Combustion Air Fan	250 m³/min x 22 kW

	Item	Unit	Value
Stack Gas (Chimny)	Flow Rate Nm³/h		13,000
	Temperature °C		22
	Vapor Volume %		5.1
	Particulate Smission g/Nm³		0.16~0.18
	SO_x	ppm	2 ~ 12
	HCl	ppm	20
	Cl_2	ppm	2.2 ~ 2.7
	NO_x	ppm	10 ~ 19
	CO_2	%	4.0
	O_2	%	16.0

	Item	Value
Heavy Oil	Feed Rate ℓ/h .	49
	Temperature °C	50

	Item	Value
Ash .	Volume kg/h	300
	Temperature °C	100
	Ignition Loss Wt. %/Solid	3.46
	Sulfur Weight %	3.62
	Absolute Specific Weight	3.14

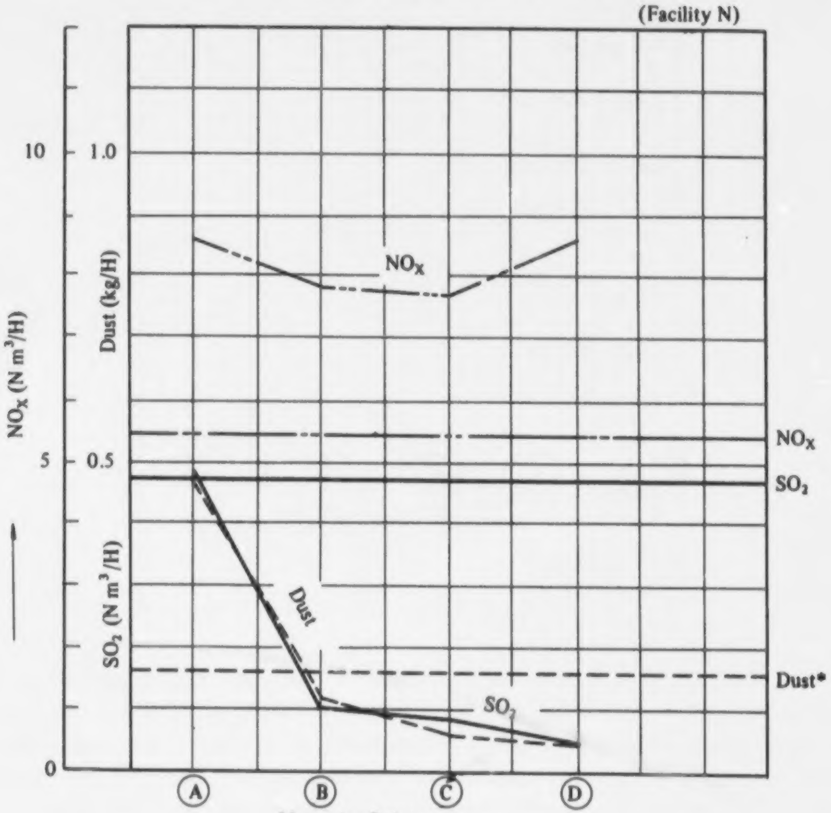
	Item	Value
Stack Gas (Furnace Outlet)	Temperature °C	290
	Vapar Volume %	36.6
	Particulate Emission g/Nm³	1.12 ~ 1.46
	SO_x ppm	115 ~ 123
	HCl ppm	92 ~ 99
	Cl_2 ppm	1.2 ~ 1.3
	NO_x ppm	58 ~ 62
	CO_2 %	9.4
	O_2 %	9.2

	Item	Value
Stack Gas (Scrubber Outlet)	Flow Rate Nm³/h	10,200
	Temperature °C	15
	Vapor Volume %	2.2
	Particulate Emission g/Nm³	0.11
	SO_x ppm	4 ~ 12

	Item	Value
Dust	Sulfur Weight %/solid	3.95
	Absolute Specific Weight	2.63

The quantities of SOx, dust and NOx at measuring points per hour, have been plotted in Fig. 3.5. Since it is not yet settled standard of concentrations of NOx emissions for sewage sludge incinerator, the measured concentration were compared with standards applied to control emission of NOx for boilers. Results showed that NOx values exceeded this reference values.

Fig. 3.5 Flow Rate of Pollutants (Facility N)



The efficiency of the cooling tower in the processing of gas emissions was quite evident, but because of the low concentrations of pollutant in the gas phase, it was not clear whether the cleaning device with chemicals was so effective in removing these pollutant.

(2) A P.O.S.T.P.

This plant treates 43,000 m³/day of sewage, and serves a population of 35,000 people. Activated sludge (step aeration) are adopted for treatment. The multiple furnace can handle 40 wet tons/day. The operating condition are listed in Table 3.16. The gas emission control equipment is shown in Fig. 3.6, and the results of analysis of the emissions summarized in Table 3.17.

Table 3.15 Performance of Incinerator in N S.T.P.

Type	Multiple Hearth Furnace	Remark
Design Capacity	Fu. No. 1 30 Ton/Day Fu. No. 2 50 Ton/Day	
Dewatering	Fp · Va	
Chemical Conditioning	Ca $(OH)_2$ + $FeCl_3$	
Running Time (Hr/Day)	Fu. No. 1: 21 Fu. No. 2: 3	
Keeping Warm Time (Hr/Day)	Fu. No. 1: 3 Fu. No. 2: 21	
Cake Moisture (%)	Va (Fu. No. 1): 74 Fp (Fu. No. 2): 63	
Incinerated Cake (Ton/Day)	Fu. No. 1 22.40 Fu. No. 2 4.20	
Aux. Fuel (A Heavy Oil) (L/Day)	3,293 Fu. No. 1 1,805 Fu. No. 2 1,488	
Kerosine (L/Day)	–	
Electricity (kWH/Day)	4,872	
Absorption Liquor (L/Day)	NaOH (Concentration 48%) 208 NaClO (Available Chlorine 12%) 305	
Water (M^3/Day)	1,920	

The soot and dust level at the incinerator outlet was found to be 0.086 g/Nm^3, which is much lower than the 1 to 3 g/Nm^3 which normally reported in past data. This result is attained by which characteristics of applied sludge are stable and making it possible to operate continually with internal pressures (in the incinerator) of –1 to –1.5 mmH_2O. And this would reduce the level of dust which is found in the gas emissions. Such stable operating conditions reflect careful and responsible plant management.

Similar to the dust levels, there was no problem with SO_2 levels either, since levels inside the incinerator were already below the standard emission level. In order to remove offensive odor, after burner fuel using kerosene. Emission gas was heated to 750°C, and although there was a slight increase in SO_2 level after the deodorizer stage, analysis of offensive odor gave results which were even better than the performance the plant at city N.

NOx levels increased after the after burning.

Operating costs of both incinerators are outlined in Table 3.18. The figures indicate that the facilities in A P.O.S.T.P cost about 10 times as much as the facilities in N P.O.S.T.P. A major reason for the higher cost is the cost of kerosene used in the after burning.

Table 3.16 Performance of Incinerator in A S.T.P.

Type	Multiple Hearth Furnace	Remark
Design Capacity	40 Ton/Day	
Dewatering	Va	
Chemical Conditioning	$Ca(OH)_2 + FeCl_3$	
Running Time (H/Day)	15	
Keeping Warm Time (Hr/Day)	9	
Cake Moisture (%)	83	
Incinerated Cake (Ton/Day)	21.01	
Aux. Fuel (A Heavy Oil) (L/Day)	2,033 { Incineration 1,676 Keeping Warm 357	
Kerosine (L/Day)	3,795	For After Burner
Electricity (kWH/Day)	2,044	
Absorption Liquor (L/Day)	NaOH (Concentration 48%) 50	
Water (M^3/Day)	1,200 { Cooling 1,056 E.P Spray 144	

Fig. 3.6 Flow Diagram of Facility A

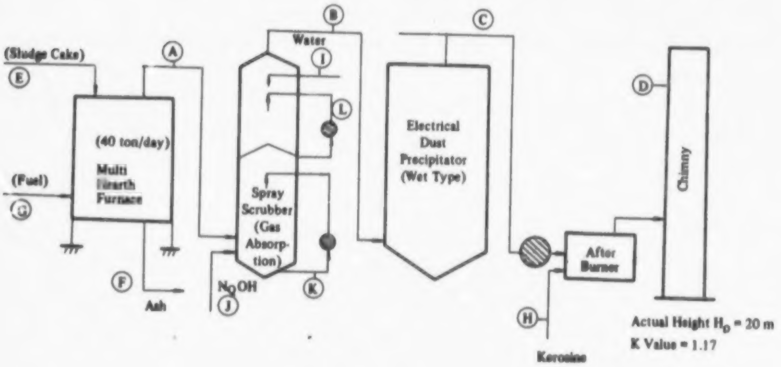


Table 3.17 Performance Data in A S.T.P.

(Nov. 5, 1976)

Item		Unit	E: Cake	F: Ash	G: A Heavy Oil	H: Kerosine
Feed Cake or Discharged Ash		kg/h	1,620	123		
Aux. Fuel		ℓ/h			113	270
Composition Analysis of Feed Cake or Discharged Ash	Moisture	%	82.9	0.5		
	Free Moisture	%	9.0	1.1		
	Ash Content	%	46.4	95.4		
	Sulfur Content	%	0.9	1.5		
	Calorific Value	KCal/kg	2680	50		

Item	Unit	A	B	C	D	Standard
Gas Temperature	°C	281	28	22	465	
Dry Gas Flow Rate	Nm^3/h	4,980	4,970	5,190	8,260	
Vapor Flow Rate	Nm^3/h	1,310	170	140	1,240	
Wet Gas Flow Rate	Nm^3/h	6,290	5,140	5,330	9,500	
Vapor Volume	%	20.8	3.4	2.6	13.0	
Particulate Emission	g/Nm^3	0.086	0.036	0.008	0.006	Less than 0.2
SO_2	ppm	81.7	10.4	9.5	16.0	Less than 84
NOx	ppm	126	119	112	144	
HCl	ppm	52.6	25.1	24.1	9.2	
Cl_2	ppm	0.17	0.95	1.04	0.15	
Offensive Odor	–	No Data	No Data	No Data	No Data	
Ammonia	ppm	61.96	2.80	2.09	2.21	
Trimethylamine	ppm	0.013	0.007	0.006	0.010	
Hydrogen Sulfide	ppm	0.095	0.100	0.068	0.017	
Methyl Sulfide	ppm	0.611	0.003	0.003	0.002	
Methylmercaptan	ppm	0.072	0.069	0.049	0.011	
Methyl Disulfide	ppm	0.007	<0.001	<0.001	0	
Acetaldehyde	ppm	0.020	0.006	0.005	<0.001	
Styrene	ppm	0.198	0.194	0.190	0.091	
O_2	%	13.8	13.9	14.4	10.5	
CO_2	%	5.5	5.4	5.2	7.8	

Table 3.18 Cost Comparison of Incinerators in N & A S.T.P. (in 1976)

Item	N S.T.P.	A S.T.P.	Remark
Fuel	2,474 ¥/h (72.75ℓ/h × 34 ¥/ℓ)	3,842 ¥/h 113ℓ/h × 34 ¥/ℓ)	34 ¥/ℓ (A Heavy Oil)
Electricity	1,560 ¥/h (156 kWH × 10 ¥/kWH) Emission Control 700 ¥/h	1,470 ¥/h (147 kWH × 10 ¥/kWH) Emission Control 960 ¥/h	10 ¥/kWH
Water	720 ¥/h (80 m^3/h × 9 ¥/m^3)	450 ¥/h (50 m^3/h × 9 ¥/m^3)	9 ¥/m^3 (Treated Effluent)
Absorption Liquor	108 ¥/h (NaOH 2.4 ℓ/h × 45 ¥/ℓ) 875 ¥/h (NaOCl 50 ℓ/h × 17.5 ¥/ℓ)	153 ¥/h (NaOH 3.4 ℓ/h × 45 ¥/ℓ)	NaOH 45 ¥/ℓ NaOCl 17.5 ¥/ℓ
Fuel (Afterburning)		10,560 ¥/h (264ℓ/h × 40 ¥/ℓ)	40 ¥/ℓ (Kerosine)
Total	5,737 ¥/h	16,475 ¥/h	
Costs, ¥/Ton-W.S.	1,887	10,170	
Costs, ¥/Nm^3-W.G.	0.2	1.9	

3.5 TENTATIVE PROPOSAL FOR STANDARD EMISSION CONTROL SYSTEM IN SLUDGE INCINERATORS

When considering a standard emission control system of gas emissions, it is very difficult to estimate emission conditions quantitatively because of the considerable degree of variation in both amount and composition of the exhaust gases being generated. Emission standard also vary according to location. Summary of the presently available information on the fludized bed incinerator, and the more common multiple furnaces, are presented in Fig. 3.7 and Table 3.19. In Fig. 3.7, A refers to standard control systems, while B refers to cases subject to much more stringent controls.

The temperature of the emission gas at the incinerator outlet is normally around 150 to 300°C, so there is little advantage in heat recovery. Because of the 1 to 3 g/Nm^3 dust levels at outlet of furnace, a dust removal is incorporated in the cooling facility, thus alleviating the load of the following washing tower with chemical or electric precipitator.

Fig. 3.7 Flow Rate of Pollutants

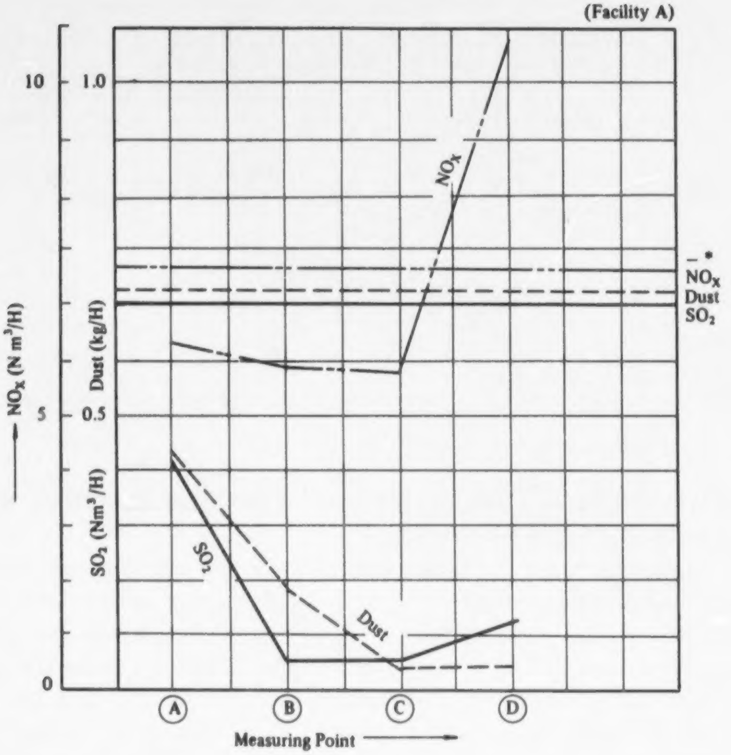


Fig. 3.8 Basic System for Emission Control

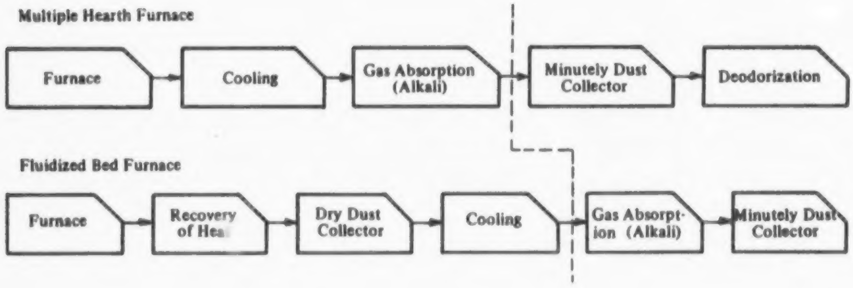


Table 3.19 Standard Requirement of Emission Control Equipment for Two Different Furnaces

	Multiple Hearth Furnace	Fluidized Bed Furnace
Heat Recovery	Stack gas temperature ranges from 150~300°C. It is too low to recover the heat.	Stack gas temperature ranges from 800 to 900°C. Excess heat above 300°C can be recovered.
Dry Type Dust Collector	Ash is disposed from bottom pit. Dust concentration in stack gas is as low as 1~3 $g/N \cdot m^3$ and, no dust removal is required, unless wet scrubber follow.	All ash is included in stack gas. Dry type dust collector is required to reduce load for wet type scrubber.
Cooling	Cooling is required as pretreatment in advance to absorption or minute dust collection.	
Absorption	SOx, HCl and Cl_2 should be removed through absorption process.	
Minutely Dust Collection	Fine particle material should be removed to attain such a low concentration as 0.05 $g/N \cdot m^3$.	
De-odorization	Deodoring is required.	Operation temperature is high enough to decompose all smelling material. No deodoring equipment is required.

In the next stage, the absorption and removal of SOx, HCl, chlorine (Cl_2) and other acidic gases conducted by washing with chemical. The SOx level at the incinerator outlet is normally 20 to 500 ppm. Not all of the sulfur compounds in the dewatered cake is transfered into the emission gas, some 50 to 80% remains in the ash. This is because of the presence on non-combustible sulfates (like $CaSO_4$ and $BaSO_4$) etc. in the dewatered cake, and because of the likely reaction between SOx (produced during incineration) and alkaline components in the ash. Previous surveys found that water washing removed large proportions of SOx, but to meet stringent emission standard, caustic soda (NaOH) washing needs to adopt. But since dust still remain in the gas at this stage, and higher pH levels, although reducing the SOx level in the emission induce, the caustic soda to react with the CO_2 in the gas emission, the pH is regulated to about 7 to 8, employing a recycling method. Excessive consumption of NaOH, and blockage of filter due to the dust, are thus avoided. HCl and Cl_2 are also removed in the same process, HCl being sufficiently reduced by water or alkaline solution washings, and Cl_2 absorbed by the alkaline solution.

Most of the dust is removed by cooling and absorption, these processes reducing the concentration down to about 0.2 to 0.5 g/Nm^3. However, these processes are not very efficient in removing the very fine particles which diameter are less than 10μ. Low pressure difference mechanical dust collector (Cyclon) are not very efficient either, but fine dust collector (electric dust precipitator etc.) can bring the level down to about 0.05 g/Nm^3.

The dilution ratios in odor measurements under normal conditions varied considerably from 70 to 20,000. Although after burning is used to remove offensive odor, this process consumes large amounts of fuel. Therefore heat recovery is necessary. Chemical washings with acid and alkalis are also reasonably efficient in reducing offensive odor, but they cannot achieve complete removal.

Concentration of nitrogen oxides (NOx) is normally about 30 to 150 ppm, and this is considered to be due to "fuel NOx" rather than "thermal NOx". The amount of nitrogen in the volatile matter of sewage sludge is 4 to 8% (by weight), 1 to 4% of which forms NOx. At the moment, however, there is no NOx emission control standards for sewage sludge incineration.

The emission gas temperature at the incinerator outlet in fluidized bed incinerators is 800 to 900°C, thus possible heat recovery up to about 300 to 350°C. However, since all the incineration ash is theoretically carried by the emission gas, special care must be exerted to prevent blockage in the heat exchanger.

The quantity of dust varies considerably according to the condition of the incinerated sewage sludge (ash contents), but normal concentration is from 30 to 160 g/Nm^3. Therefore, a dry-type dust collection is adopted (mechanical scrubber) after the heat exchanger, and this in turn is followed by a wet washing to further reduce the dust.

Furthermore, the area of contact between the sewage sludge and gas in the fludiged bed incinerator is relatively large, so when the internal temperature is maintained at 80o to 900°C, and lime used as a coagulant, a certain amount of reaction between the lime and SOx, HCl, and Cl_2 can be expected. The source and mechanism of the generation process of pollutants in gas is essentially the same as in the multiple furnace, so, the emission control system are composed with dry type dust collection.

By combustion and decomposition of offensive odor substances in the incinerator, there are no odor substances left in the emission gas, so no deodoring devices are required.

It has been believed that generation of NOx by fluidized bed incinerator had been higher than in multiple furnace, but the results of this survey show that this was not exact. There is no emission standard of NOx which is applicable to sludge combustion incinerator at the present.

CHAPTER 4. STUDIES ON SEWAGE SLUDGE PYROLYSIS

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4. STUDIES ON SEWAGE SLUDGE PYROLYSIS

4.1 INTRODUCTION

Sewage treatment plants in large municipalities and their environs are being subjected to increasing restrictions on the acquisition of land for sewage sludge landfill in Japan. For this reason, there are found many cases where sludge is incinerated at sewage treatment plant sites for the most effective use of the land acquired to their sludge filling.

However, the incineration of sewage sludge has been faced with several severe problems, which require an early settlement.

First, the more severe environmental quality standard on air pollution has established it indispensable for sewage treatment plants to set up incidental facilities of incinerators, such as electric precipitators, to prevent pollution causing by the exhaust gas. The construction and operation of such facilities require additional expenses.

Second, it has also become necessary to set up incidental facilities of incinerators for offensive odor control, such as exhaust gas combustion (afterburning) facilities, because of the strong demand from residents for preventing offensive odor arising from sludge incineration. The construction and operation of such facilities also require additional expenses.

Third, it has been found that lime used as a conditioner of sludge dewatering, and trivalent chromium compounds contained in municipal sewage sludge show a chemical reaction in the furnace at the time of incineration. The trivalent chromium compounds in sludge are oxidized into hexavalent chromium compounds which are soluble in water. Therefore, the filling of land with incinerated ash has a danger of causing the pollution of surface water and underground water by hexavalent chromium compounds. For the prevention of such pollution, it requires much expense for landfill.

Fourth, the operational cost for sludge incineration – fuel cost and electric power charge – has been increasing sharply every year due to the spirals in the prices of oil products in addition to the increased use of fuel and electric power for incidental facilities of incinerators. Sewage treatment plants in some areas are obliged to use better quality fuel than before to establish the environmental quality standard on air pollution.

Under these circumstances, it is expected to be more advantageous, for the following reasons, to reduce the volume of solid residue (ash) by pyrolysis than to incinerate it.

i) In pyrolysis, the furnace is operated naturally at the lower combustion air ratio than in incineration. Accordingly, the amount of exhaust gas is reduced and the construction cost and the operation cost (fuel cost and electric power charges) of the incidental facilities for the control of both air pollution and offensive odor might be able to cut down.

ii) Since the inside of the pyrolysis furnace is maintained in a reducing condition or in a condition close to it, there is little possibility of the trivalent chromium compounds in sewage sludge being oxidized into haxavalent chromium compounds.

iii) Pyrolysis might require more fuel in the furnace itself than incineration. However, the amount of fuel used in the furnace might be able to reduce, if combustible gas obtained by pyrolysis is burned in the combustion chamber and steam generated in boiler is used for preliminary drying of sludge. Therefore, there is expected a strong possibility that fuel consumption for the pyrolysis as a whole system could be smaller than that for incineration.

So far, multi-hearth furnace have dominantly been used for sewage sludge incinerations at sewage treatment plants in Japan. Therefore, there are many skillful engineers and operators for design and operation of such furnaces. If the multi-hearth furnaces now in use are rebuilt for pyrolysis of sewage sludge, it will be expected of great benefit to the social and economical aspects in the nation.

For these reasons, the Ministry of Construction decided on the disbursement of the public works technology and development subsidy for fiscal 1976 to conduct practical study work on pyrolysis by applications of multi-hearth furnace. The ministry offered for public subscription to interesting organizations and, after screening application documents, decided to give the subsidy to a privated firm (NGK Insulators, Ltd., Nagoya). The company, which has experiences of basic study works in the past (1), proposed to conduct additional basic study and new practical study at pilot plants. The Ministry of Construction organized a discussion group to have the views of users reflected on the study. The group is made some engineers of Kyoto University, Osaka Prefecture, Kyoto City, Hyogo Prefecture and the Japan Sewage Works Agency as well as the Ministry of Construction.

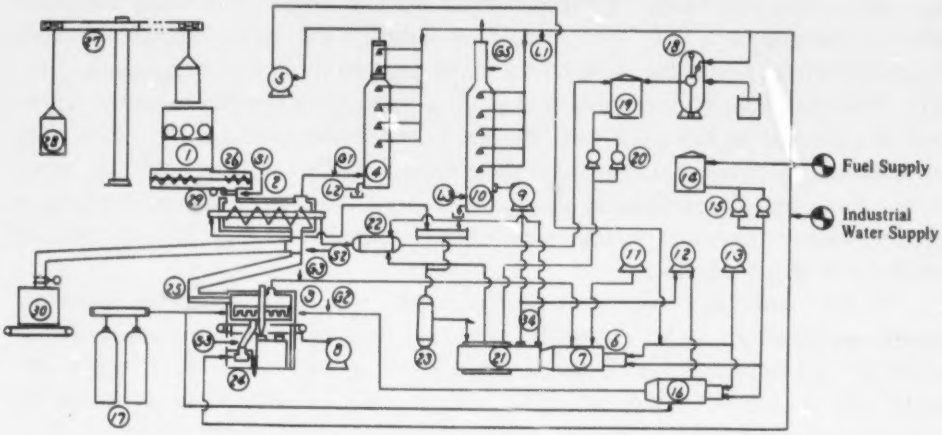
The Practical study was not completed at the time of this reporting. However, a series of experiments were concluded and their results were compiled. They were conducted at a pilot plant established in the Kawamata Sewage Treatment Plant site of Osaka Prefecture, using raw dewatered sludge carried from the Toba Sewage Treatment Plant in Kyoto. At present, the project is being examined from the economic viewpoint, and conditions for the designing of facility to be put to practical use are being discussed.

More experiments are to be conducted at the pilot plant, using heat-treated sludge taken from a sewage treatment plant in Osaka Prefecture and raw dewatered sludge separated from waste at a chromic tanning pre-treatment facility in Hyogo Prefecture.

4.2 PILOT PLANTS AND SLUDGE CAKES USED IN EXPERIMENTS

Basic study and practical study were conducted separately at different pilot plants. Used in the basic experiments was a pilot plant with a capacity to treat 2 tons of sludge cakes per day with the water content of 75% The flow chart of this pilot plant is shown in Fig. 4.1. The Figure shows the sampling points for solid ($S_1 - S_3$), liquid ($L_1 - L_3$) and gas ($G_1 - G_5$) in addition to various sorts of machinery and instrumentation constituting the pilot plant.

Fig. 4.1 Pilot Plant Flow Chart Used for Basic Study



Explanation

①	Cake feeder	⑥	Burner	⑪	Blower for combustion chamber	⑯	Heat source chamber	㉑	Exhaust gas boiler	㉖	Conveyer
②	Paddle dryer	⑦	Combustion chamber	⑫	Blower for circulated gas	⑰	Nitrogen gas supply	㉒	Condensed water tank	㉗	Hoist
③	Single-hearth furnace	⑧	Blower for partial combustion	⑬	Blower for burner	⑱	Water softening installation	㉓	Condensed water circulator	㉘	Cake container
④	De-humidifer	⑨	Blower for exhaust gas	⑭	Fuel tank	⑲	Soft water storage tank	㉔	Solid residue cooling tank and container	㉙	Double sealing damper
⑤	Blower for dryer	⑩	Scrubber stuck	⑮	Fuel pump	⑳	Water supply pump	㉕	Screw type feeder	㉚	Dried cake container

Sampling point

Sampling point of solids

S_1 Cake of dryer inlet
S_2 Cake of dryer outlet
S_3 Residue of pyrolysis

Sampling point of liquids

L_1 Supplied water
L_2 Wastewater of dehumidifer
L_3 Wastewater of scrubber

Sampling point of gases

G_1 Outlet of dryer
G_2 Outlet of pyrolysis furnace
G_3 Inlet of combustion chamber
G_4 Outlet of exhaust gas boiler
G_5 Outlet of scrubber

The flow chart of the pilot plant used for practical study is shown in Fig. 4.2. It is different from the pilot plant for the basic study in the following points.

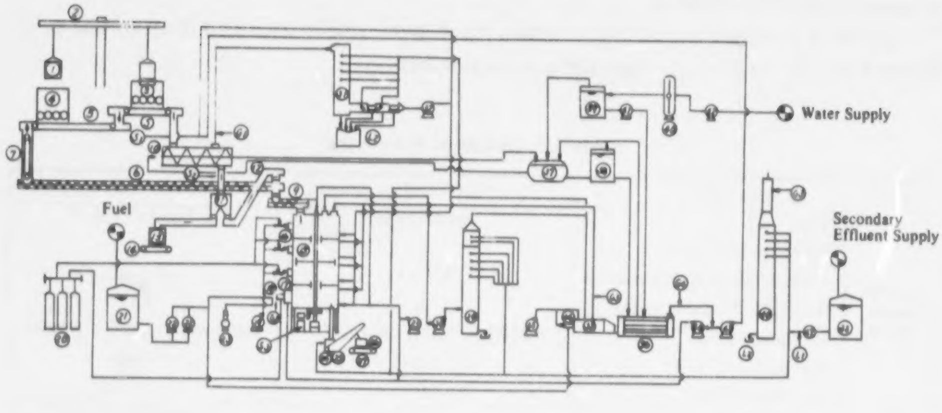
i) The indirect steam dryer of paddle type was changed from a two-shaft dryer to a four-shaft dryer.

ii) The plant was designed so that dewatered sludge cakes could also be thrown into the furnace directly without passing through the indirect steam dryer of paddle type.

iii) The single-hearth furnace was changed to a four-hearth furnace.

iv) The heat source chamber was directly attached to the pyrolysis furnace in

Fig. 4.2 Pilot Plant Flow Chart Used for Practical Study



Explanation

①	Cake container	⑪	Sieve machine	㉑	Fuel tank	㉛	No. 1 scrubber	㊶	Water supply tank
②	Hoist	⑫	Dried cake conveyer	㉒	Fuel pump	㉜	Blower for dryer	㊷	No. 2 water supply pump
③	Cake feeder	⑬	Dried cake container	㉓	Compressor	㉝	Combustion chamber	㊸	No. 1 blower for exhaust gas
④	Dried cake feeder	⑭	Roller conveyer	㉔	Quenching tank	㉞	Burner for combustion chamber	㊹	Blower for circulation
⑤	No. 1 cake conveyer	⑮	Pyrolysis furnace	㉕	Residue carry conveyer	㉟	Blower for combustion chamber	㊺	No. 2 scrubber
⑥	No. 2 cake conveyer	⑯	Ignition burner for partial combustion	㉖	Residue container	㊱	Boiler	㊻	Water storage tank
⑦	No. 3 cake conveyer	⑰	Heat source chamber	㉗	Roller conveyer	㊲	Condensed water tank	㊼	No. 1 water supply pump
⑧	No. 1 screw conveyer	⑱	Heat source burner	㉘	Blower for pyrolysis	㊳	Condensed water circulator		
⑨	No. 2 screw conveyer	⑲	Blower for burner	㉙	No. 3 scrubber	㊴	Soft water tank		
⑩	Paddle dryer	⑳	Propane gas supply	㉚	No. 2 blower for exhaust gas	㊵	Water softening installation		

Sampling point

Sampling point of solids	Sampling point of liquids	Sampling point of gases
S_1 Cake of dryer inlet	L_1 Supplied water	G_1 Outlet of dryer
S_2 Cake of dryer outlet	L_2 Wastewater of No. 1 scrubber	G_2 Outlet of heat source chamber
S_3 Residue of pyrolysis furnace	L_3 Wastewater of No. 2 scrubber	G_3 Outlet of pyrolysis furnace
		G_4 Outlet of combustion chamber
		G_5 Outlet of scrubber

order to reduce heat loss.

v) The pilot plant was designed so that experiments could be conducted with the drying-pyrolysis process as a continuous system.

vi) The system to deal with solid residue discharged from the furnace was changed so that it would drop into water to be scooped up automatically.

vii) A propane gas burner was attached to each hearth of the pyrolysis furnace as a supplemental heat source.

viii) The combustion chamber was designed to allow the automatic control of oxygen concentration in it.

Table 4.1 shows the details of main machinery and instrumentation of the pilot plants both for basic study and for practical study.

Table 4.1 Details of Pilot Plant

Name	Items	Pilot Plant for Basic Study	Pilot Plant for Practical Study
Indirect Steam Dryer	Type	Paddle Type	Paddle Type
	Numbers of Shaft	2	4
	Areas of Total Heat Transfer	8.1 m^2	10.1 m^2
	Numbers of Screw per Shaft	16	12
	Dimension (mm)	560 W x 850 H x 2,800 L	1,097 W x 1,125 H x 2,300 L
	Revolution of Shaft	12 rpm	15 rpm
	Counter of Drain	Volumetric	Volumetric
Pyrolysis Furnace	Method of Heating	Inner Heating	Inner Heating
	Type	Single-Hearth Furnace	4-Hearth Furnace
	Total Bed Areas	2.1 m^2	1.63 m^2
	Dimension (mm)	1,800 ID x 2,200 OD x 1,000 H	900 ID x 1,360 OD x 2,096 H
	Attachment	–	Gas Burner 5,000 kcal/hr x 3
Heat Source Chamber	Capacity of Burner	90,000 kcal/hr	80,000 kcal/hr
	Dimension (mm)	400 ID x 800 OD x 1,500 L	250 ID x 760 OD x 720 L
	Duct Length to Pyrolysis Furnace	4 m	0.5 m
	Attachment	Circulating Gas Flow Control Unit	Circulating Gas Flow Control Unit
Com-bustion Chamber	Type	Direct Combustion	Direct Combustion
	Capacity of Burner	90,000 kcal/hr	150,000 kcal/hr
	Volume of Chamber	0.2 m^3	0.25 m^3
	Dimension (mm)	400 ID x 800 OD x 1,500 L	400 ID x 800 OD x 2,412 L
Exhaust Gas Boiler	Type	Breeching 3 Pass	Breeching 3 Pass
	Areas of Total Heat Transfer	7.7 m^2	5.8 m^2
	Production of Steam	100 kg/hr	100 kg/hr
	Max. Pressure of Steam	7 $kg/cm^2 \cdot G$	9.5 $kg/cm^2 \cdot G$
	Drain Storage Tank	0.25 m^3	0.33 m^3
	Drain Circulator	0.03 m^3	0.038 m^3
	Soft Water Tank	0.2 m^3	0.2 m^3
	Softening Water Installation	200 kg/hr	200 kg/hr
	Water Supply Pump	660^2/hr x 80 mH	200^2/hr x 10 kg/cm^2

The dewatered cakes used in the experiments were taken from the vacuum filter room of the Toba Sewage Treatment Plant in Kyoto City. Table 4.2 shows the analytical results of dewatered cake samples. The compositions of ignition loss of the dewatered cake samples are shown in Table 4.3. The analytical results of the perfect ash in the dewatered cake samples are also shown in Table 4.4.

4.3 BASIC STUDY ON DRYING-PYROLYSIS PROCESS

4.3.1 EXPERIMENTS OF DRYING FOR DEWATERED CAKE USING BY INDIRECT STEAM DRYER

The dryer used in the experiments is an indirect steam dryer of paddle type shown in Fig. 4.3. Steam used for the dryer is supplied by the exhaust gas boiler ㉑. That is, after the pressure of steam is adjusted so that steam temperature will reach a fixed point, the steam is supplied to the jacket, screw and shaft of the dryer, as shown in Fig. 4.3. And, the steam is condensed into moisture after being used for drying dewatered cakes. The moisture was returned to the exhaust gas boiler after volumetric measurement. In the experiments, the Overall Heat Transfer Coefficient (U) was found by Equation-(1).

$$U = Q/A \cdot \Delta t \ (\mathrm{Kcal/m^2 \cdot hr \cdot {}^\circ C}) \quad \ldots\ldots \quad (1)$$

Where, $\Delta t = t_s - t_c$

$Q = Y_s \times M_d$

Q : Heat Transfer Capacity (Kcal/hr)

A : Total Heat Transfer Area of the Dryer (m^2)

t_s : Temperature of Supplied Steam (°C)

t_c : Average Temperature of Dewatered Cakes in the Dryer (°C)

M_d : Amount of Condensed Moisture Generated (kg/hr)

Y_s : Condensed Heat of Supplied Steam (Kcal/kg)

Some examples of relationship between the moisture content (%) of raw sludge cakes at the inlet of the dryer and the Overall Heat Transfer Coefficient (U) is shown in Fig. 4.4. There was no major difference in drying efficiency between raw sludge and digested sludge, and their Overall Heat Transfer Coefficients (U) are within the range of 70 ~ 100 $\mathrm{Kcal/m^2 \cdot hr \cdot {}^\circ C}$. Similar experiments on heat-treated sludge showed that its Overall Heat Transfer Coefficient (U) was 215 $\mathrm{Kcal/m^2 \cdot hr \cdot {}^\circ C}$. This is presumably because the fullness ratio of heat-treated sludge in the dryer is higher than that of raw sludge or digested sludge and because moisture contents in dewatered cakes exists in different conditions.

Fig. 4.3 Schematic Drawing of the Indirect Steam Dryer of Paddle Type Used for Pre-Treatment of Sludge Cake

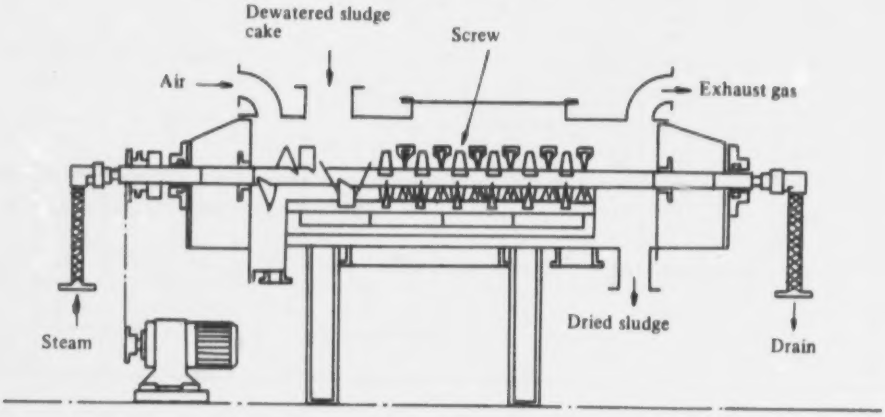


Fig. 4.4 Relationship between Moisture Content and Overall Heat Transfer Coefficient

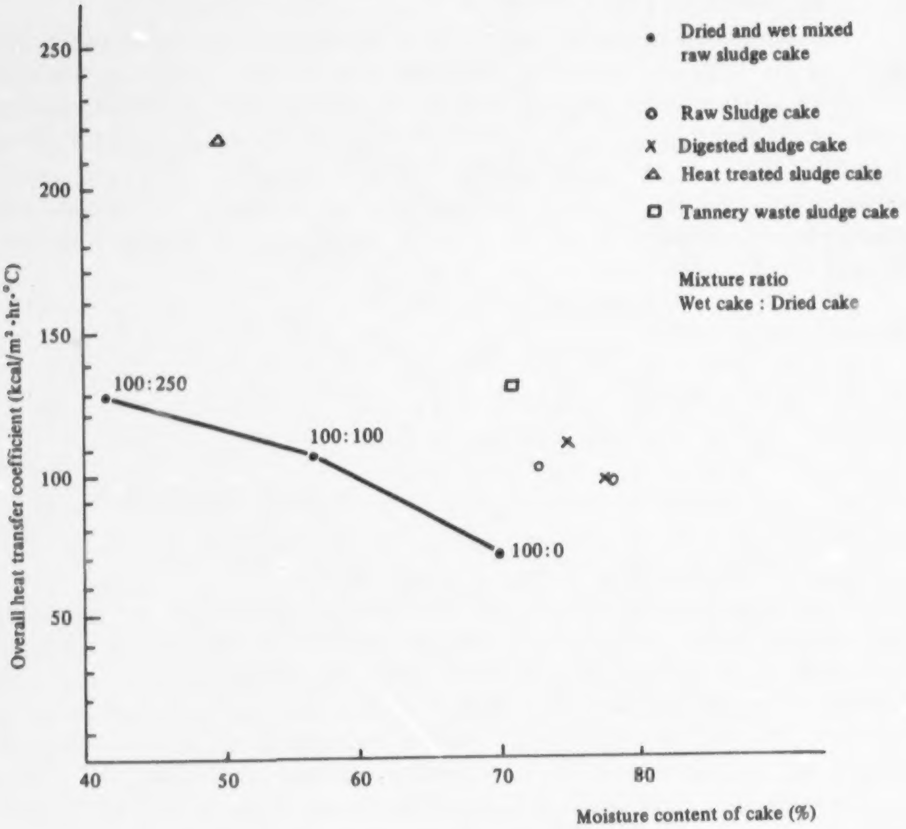


Table 4.2 Analytical Result of Dewatered Cake Samples

(Dry Base)

Date of Sampling	Moisture Content (%)	Ignition Loss (%)	Volatile Content (%)	Gross Calorific Value (kcal/kg-DS)	C (%)	H (%)	N (%)	S (%)	O (%)	Cl (%)	CN (ppm)	NH_3 (ppm)	Hg (ppm)	Cd (ppm)
Aug. 9	74.6	55.0	53.2	2,470	24.8	4.1	2.8	0.71	25.2	–	–	–	–	–
Aug. 30	75.6	49.3	52.0	1,800	18.0	4.0	2.9	0.51	30	1.0	4	220	1.7	7.2

Date of Sampling	Pb (ppm)	As (ppm)	Total Cr (ppm)	Cr^{+6} (ppm)	Zn (ppm)	Cu (ppm)	Fe (ppm)	Al_2O_3 (%)	CaO (%)	MgO (%)	Na_2O (%)	K_2O (%)	SO_3 (%)
Aug. 9	100	–	200	–	2,600	–	–	4.8	14.08	–	–	0.49	13.88
Aug. 30	175	4.4	170	7.3	1,450	360	6.21	2.17	26.3	0.77	0.18	–	7.98

Table 4.3 Compositions of Ignition Loss in Samples

Date of Sampling	Gross Calorific Value (kcal/kg·DS)	C (%)	H (%)	N (%)	S (%)	O (%)	Cl (%)	CN (ppm)	NH_3 (ppm)
Aug. 9	4,491	45.1	7.5	5.1	1.3	45.8	–	–	–
Aug. 30	3,651	36.5	8.1	5.9	1.0	60.9	2.4	8.1	446

Table 4.4 Analytical Result of Perfect Ash in Samples

Date of Sampling	Total Cr (T-Cr) (ppm)	Cr^{+6} (ppm)	Cr^{+6}/T-Cr (%)	Soluble Cr^{+6} ($S\text{-}Cr^{+6}$) (mg/ℓ)	Soluble Rate of Cr^{+6} (%)		CaO (%)
					$\frac{S\text{-}Cr^{+6}}{T\text{-}Cr^{+6}}$ (%)	$\frac{S\text{-}Cr^{+6}}{T\text{-}Cr}$ (%)	
Aug. 9	360	82	17.2	4.5	72.6	22.5	31.3
Aug. 30	355	190	53.5	0.36	1.9	1.0	51.9

4.3.2 EXPERIMENTS ON PYROLYSIS OF DRIED CAKES

In the experiments, dried cakes by the indirect steam dryer were stored in the feeder at first and then a constant amount of them was fed into the single-hearth furnace ③ by a screw feeder ㉕ shown in Fig. 4.1. The heat sources to the furnace were hot blast from the hot source chamber and heat of partial combustion proportionate to the ratio of fixed combustion air to sludge feed to furnace. Gas from the outlet of the furnace was sent to the combustion chamber ⑦, where combustibles was burned and such offensive odor components, HCN and NH_3 were decomposed, and then the heat was recovered as steam by the waste heat boiler ㉑. Part of the exhaust gas in the boiler was circulated to adjust hot blast in which oxygen concentration is almost zero.

The remainder of the waste gas was released into the air after being sent through the scrubber ⑩. The solid residue taken out of the pyrolysis furnace was cooled indirectly without contact with the air.

The conditions of these experiments were as follows.

i) Ratio of theoretical amount of combustion air to actual amount of combustion air of fed sludge: 0 ~ 2.0

ii) Temperature in pyrolysis furnace: 600 ~ 900°C.

iii) Sludge feed loading into furnace: 10 ~ 40 kg·DS/m²·hr

iv) Estimated detention time of sludge in furnace: 15 ~ 60 min.

v) Condition of decomposition of exhaust gas

Temperature in combustion chamber: 900 ~ 1,100°C

Oxygen concentration: 0 ~ 3%

The ratio of theoretical amount of combustion air to actual amount of combustion air of fed sludge (ratio of amount of combustion air) was adjusted by a partial combustion blower.

In order to evaluate the degree of decomposition of combustibles, the ignition loss of solid residue was measured and the decomposition rate of combustibles was estimated by Equation (2).

$$\text{Decomposition Rate of Combustibles (\%)}$$
$$= \left[1 - \frac{\text{Ignition Loss of Solid Residue (\%)}}{100 - \text{Ignition Loss of Solid Residue (\%)}} \div \frac{\text{Ignition Loss of Fed Cake(\%)}}{100 - \text{Ignition Loss of Fed Cake (\%)}}\right] \times 100 \quad \ldots .(2)$$

a. Ratio of Theoretical Amount of Combustion Air to Actual Amount of Combustion Air of Fed Sludge (Ratio of Amount of Combustion Air)

Three sorts of data were collected to investigate this problem. The first concerned the changes in the form of chromium in solid residue according to the ratio of amount of combustion air, which are shown in Fig. 4.5 and Fig. 4.6. Fig. 4.5 shows the results of experiments on raw sludge at the Toba Sewage Treatment Plant. Fig. 4.6 shows, for comparison, the results of experiments on sludge of chromium tanneries' waste which contained a large amount of trivalent chromium compounds. The second data concerned the relationship between the ratio of amount of combustion air and both ignition loss of solid residue and the decomposition rate for combustibles in sludge. It is shown in Fig. 4.7.

Fig. 4.5 and Fig. 4.6 show that, when the ratio of amount of combustion air is below 1.0, the ratio of Cr^{+6}/T-Cr in solid residue is smaller than that of Cr^{+6}/T-Cr in dewatered cake and that the lower the ratio of amount of combustion air, the higher is the decrease rate. Elution of hexavalent chromium from the solid residue was not observed. If the ratio of amount of combustion air is below 1.0, oxygen supplied to the pyrolysis furnace is consumed by the combustion of inflammable gas generated by pyrolysis and the combustion of carbon retained in solid residue, without causing the oxidization of new trivalent chromium compounds. In case the ratio of amount of combustion air is still lower, a part of the hexavalent chromium compounds already contained in sludge is reduced in the pyrolysis furnace by reducing gas, such as hydrogen and carbon oxide gases generated by pyrolysis.

In contrast, the ratio of Cr^{+6}/T-Cr were fairly increased in solid residues when dewatered cake was dried and incinerated completely in an electric furnace and when it was incinerated at the furnace of pilot plant with the ratio of amount of combustion air set at 2.0, comparing to the ratio of Cr^{+6}/T-Cr in dewatered cake itself. When the amount of the hexavalent chromium compounds eluted was measured by the Solubility Test Method as provided by the Ministerial Ordinance of the Prime Minister's Office, it has been found at some plants that the soluble concentrations of hexavalent chromium in solid residue exceed the limit of 1.5 mg/ℓ sometimes, when dewatered cakes were incinerated in existing furnaces. This means that a high ratio of amount of combustion air, such as about 2.0, promotes the oxidization reaction of trivalent chromium compounds and a part of them changes hexavalent chromium compounds.

According to Fig. 4.7, the higher the ratio of amount of combustion air, the further promoted is the decomposition of combustibles in sludge. This means that a high ratio of amount of combustion air promotes the combustion of carbon remaining in solid residue.

From the viewpoint of preventing the oxidization of trivalent chromium compounds in sludge, the appropriate ratio of amount of combustion air is considered to be around 0.5 ~ 0.7.

Fig. 4.5 Behaviour of Chromium in Each Solid Residue (Raw Sewage Sludge in Toba Sewage Treatment Plant)

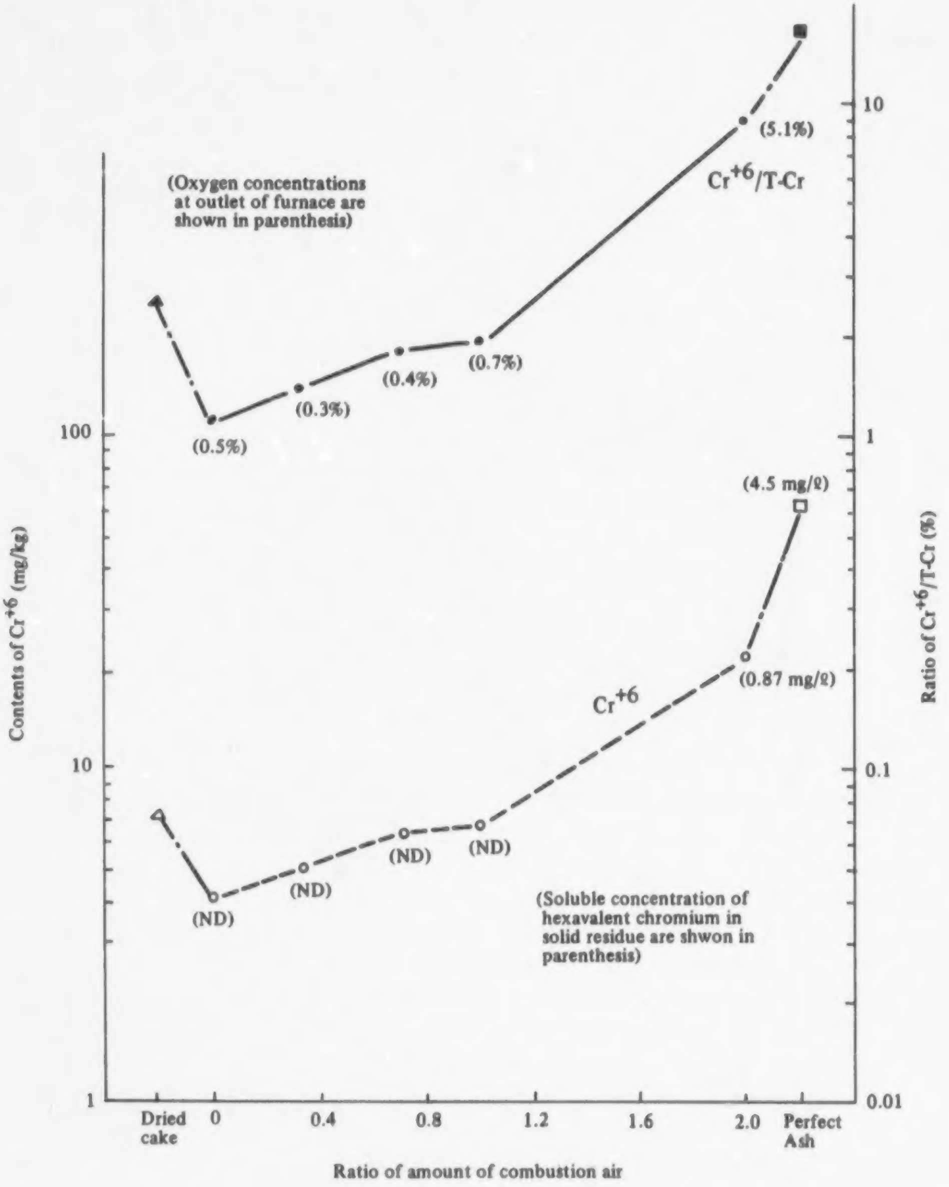


Fig. 4.6 Behaviour of Chromium in Each Solid Residue
(Raw Sludge of Tanneries Waste)

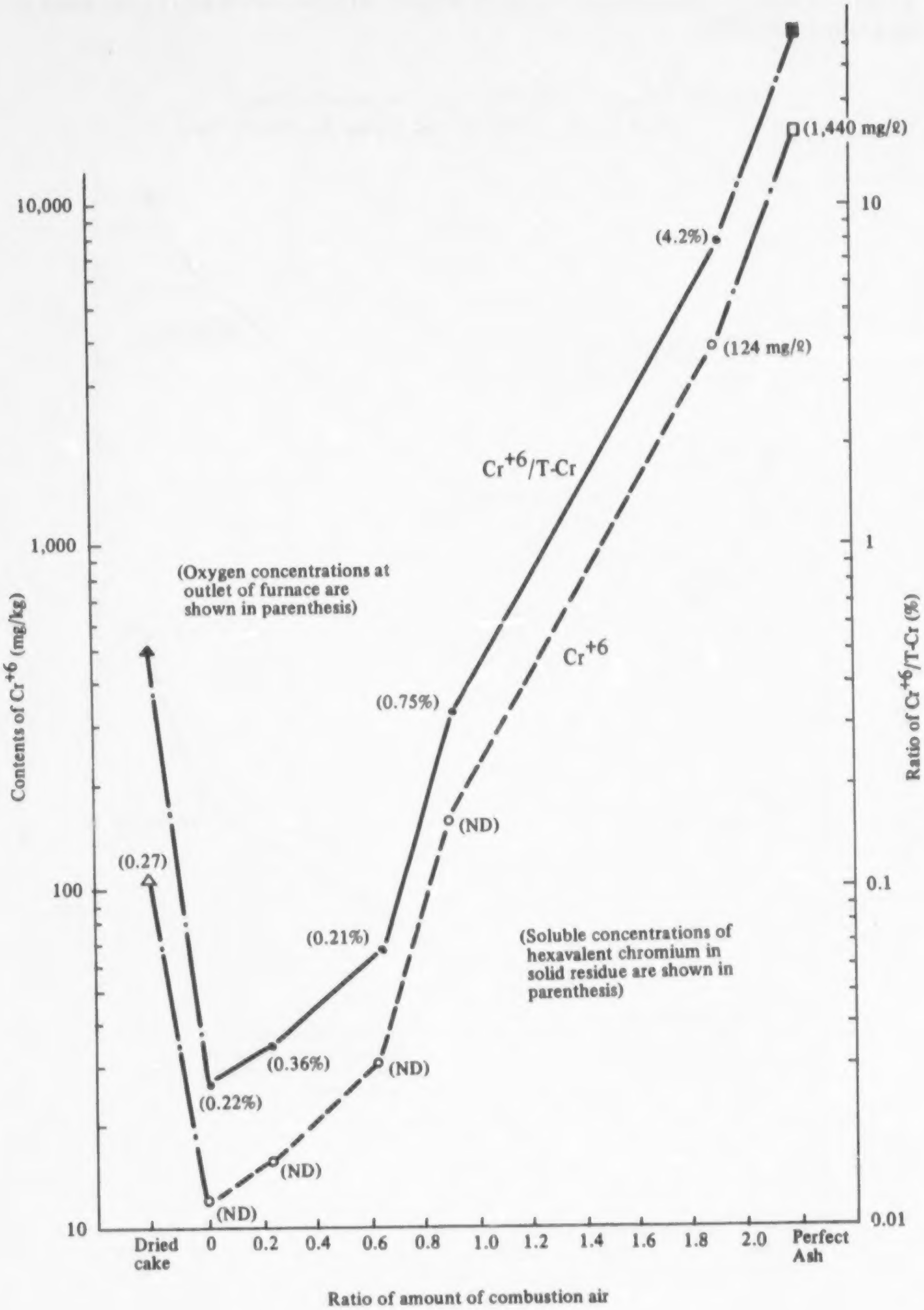
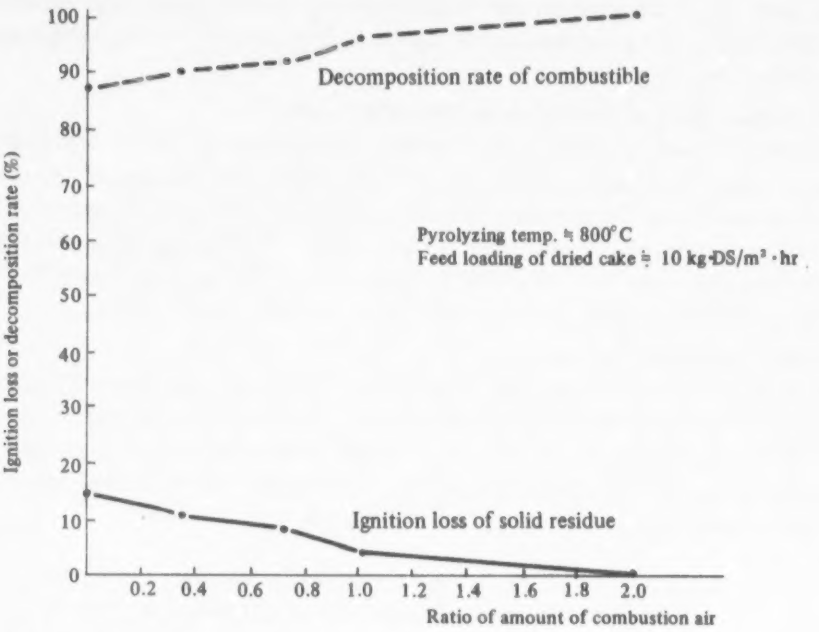


Fig. 4.7 Variation of Both Ignition Loss of Solid Residue and Decomposition Rate of Combustible to Ratio of Amount of Combustion Air



b. Pyrolyzing Temperature

Changes in pyrolyzing temperatures, ignition loss of solid residue and the rate of decomposition of comsustibles are shown in Fig. 4.8. The Figures are given for pyrolysis when the ratio of amount of combustion air was 0.6 and 0. There is a tendency that the higher the pyrolyzing temperature, the higher is the decomposition rate of combustibles and the lower is the ignition loss of solid residue. At the pyrolyzing temperature of 900°C, the decomposition of combustibles was almost completed. When the ratio of amount of combustion air was 0, the decomposition rate tended to drop by about 5% compared with pyrolysis when the ratio of amount of combustion air was 0.6.

Shown in Fig. 4.9 is the relationship between pyrolyzing temperatures and the rates of carbon, hydrogen and nitrogen remaining in solid residue. When the pyrolyzing temperature was over 800°C, most of the ignition loss of solid residue was carbon. There is a difference of more than 5% in carbon content in solid residue between pyrolysis when the ratio of amount of combustion air was 0 and when the ratio was 0.6. There was also a slight difference each in the nitrogen and hydrogen contents in solid residue according to the differences in the ratio of amount of combustion air at 0 and 0.6.

c. Feed Loading of Dried Cake

Experiments were conducted on changing feed loading of dried cake (unit: kg·DS/m²·hr) with the pyrolyzing temperature set at 900°C and the ratio of amount of combustion air at 0.6. Fig. 4.10 shows the changes in the ignition loss of solid residue and the rate of decomposition of combustibles caused by changes in feed loading of dried cake.

With the increase in the feed loading of dried cake, the ignition loss of solid residue goes up. If the ignition loss of solid residue is to be allowed at about 10% when filling land with solid residue, the adequate feed loading of dried cake is about 25 kg·DS/m²·hr.

d. Detention Time of Dried Cake in Pyrolysis Furnace

Fig. 4.11 shows the relationship between the detention time of dried cake in pyrolysis furnace and both ignition loss of solid residue and the rate of decomposition of combustibles. Details are not clear because the data of experiments are for the detention time of 15 min. and 60 min. only. Estimating from the Figure, however, the detention time of at least 45 minutes seems to be necessary to maintain the operating condition of ignition loss of solid residue below 10%.

e. Recombustion of Exhaust Gas

Experiments were conducted to grasp the operating conditions of recombustion of exhaust gas from the pyrolysis furnace in order to burn and decompose pollutants in the gas (HCN, NH_3). As a result, it became known that, when O_2 concentration is over 1%, HCN gas can be decomposed almost completely at the combustion temperature of over 900°C and NH_3 gas at over 1,000°C.

Fig. 4.8 Variation of Both Ignition Loss of Solid Residue and Decomposition Rate to Pyrolyzing Temperature

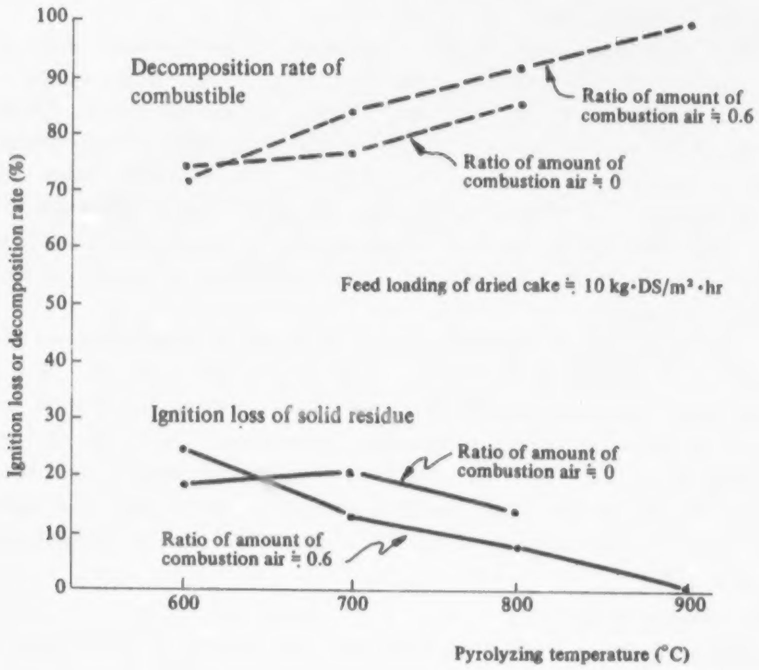


Fig. 4.9 Variation of Carbon, Nitrogen and Hydrogen in Solid Residue to Pyrolyzing Temperature and Ratio of Amount of Combustion Air

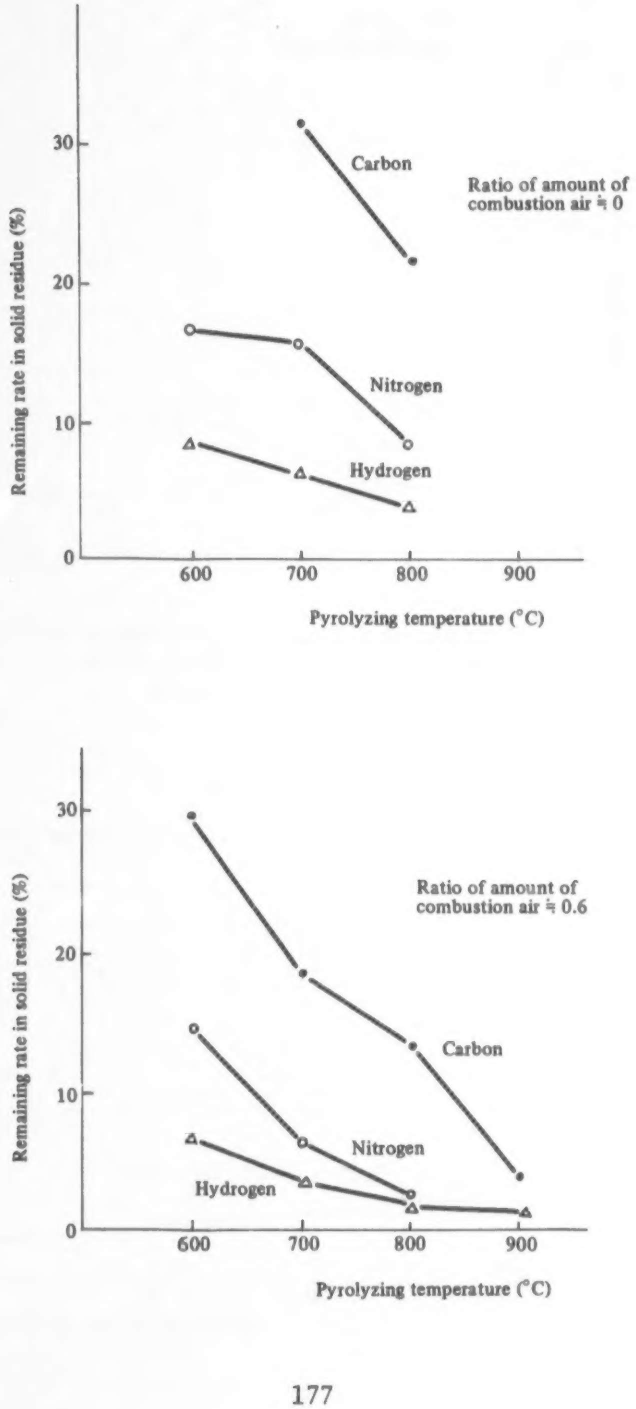


Fig. 4.10 Variation of Both Ignition Loss of Solid Residue and Decomposition Rate to Feed Loading of Dried Cake

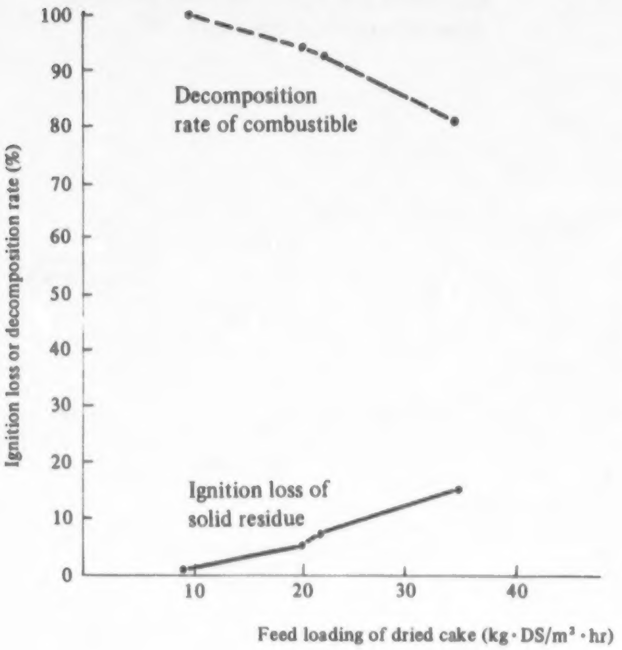
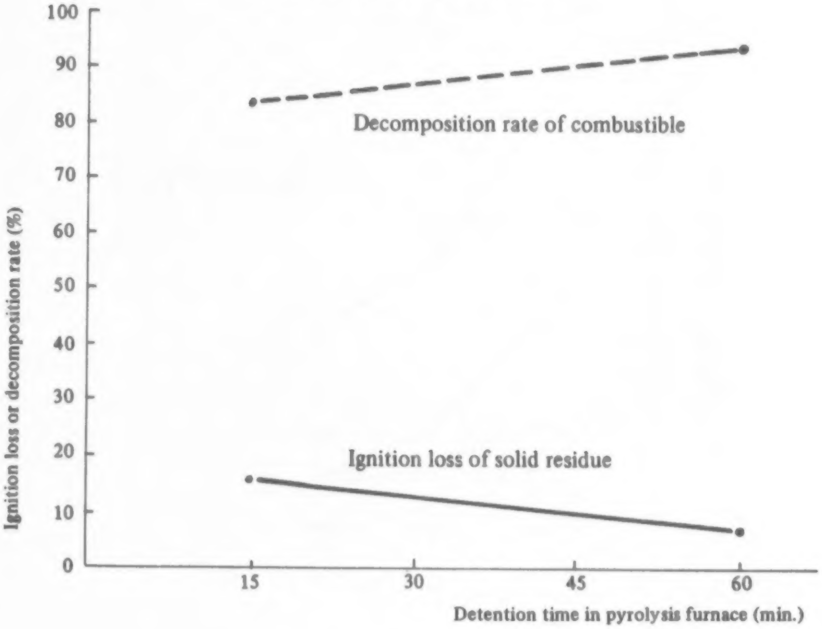


Fig. 4.11 Variation of Both Ignition Loss of Solid Residue and Decomposition Rate of Combustible to Detention Time in Pyrolysis Furnace



4.3.3 PRODUCTS OF DRIED CAKE PYROLYSIS

a. Remaining Rate of Each Element in Solid Residue

Table 4.5 shows the ratios of various elements remaining in solid residue to sludge cake. It also shows three operating conditions of pyrolysis and, for comparison, remaining rate of each element in solid residue.

Table 4.5 Remaining Rate of Each Component in Solid Residue

Run No.	Pyrolyzing Temp. (°C)	Ratio Amount of Combustion Air	Ignition Loss (%)	Gross Calorific Value (%)	C (%)	H (%)	N (%)	S (%)	O (%)	Cl (%)	Hg (%)	As (%)	Cd (%)	Pb (%)	Zn (%)	Cu (%)	Fe (%)	T-Cr (%)
1	765~800	0	23.6	14.4	30.4	6.6	9.5	95.5	39.5	65.4	<1.5	68.5	32.9	67.7	90.8	102.4	95.4	100.7
2	805~825	0.67	10.2	9.1	20.3	4.4	3.0	115.6	25.5	67.5	<1.7	77.7	28.7	53.9	113.8	125.1	82.4	135.2
3	880~910	0.75~0.77	2.8	0.9	3.8	3.9	0.5	86.7	12.1	62.2	<1.5	68.5	27.4	27.6	95.0	93.2	96.3	97.9
4	795~835	2.89	4.8	0	4.1	4.0	0	99.6	13.6	58.4	<1.5	54.2	31.7	33.3	84.3	85.4	97.5	90.6

The higher the pyrolyzing temperature and the ratio of amount of combustion air, the smaller becomes the ignition loss, thus decreasing the calorific value of solid residue. In other words, the solid residue becomes closer to that of incineration.

Analyses of six elements show that, in the case of pyrolysis, the amounts of carbon, hydrogen, nitrogen and oxygen remaining in solid residue differ, depending on pyrolyzing temperature and ratio of amount of combustion air, while the amounts of sulfer and chloride vary widely. Concerning chloride, there was a difference in amount remaining in solid residue between pyrolysis and incineration. Pyrolysis tends to vaporize less chloride than incineration.

Concerning such heavy metals as mercury, cadmium and iron, there is hardly any difference in amount remaining in solid residue between pyrolysis and incineration. But pyrolysis can slightly reduce the vaporization into the air of arsenic, zinc, copper and total chromium, compared with incineration. It was also found that the amount of lead vaporized into the air differs greatly, depending on operating conditions of pyrolysis.

b. Concentration and Particle Size Distribution of Dust

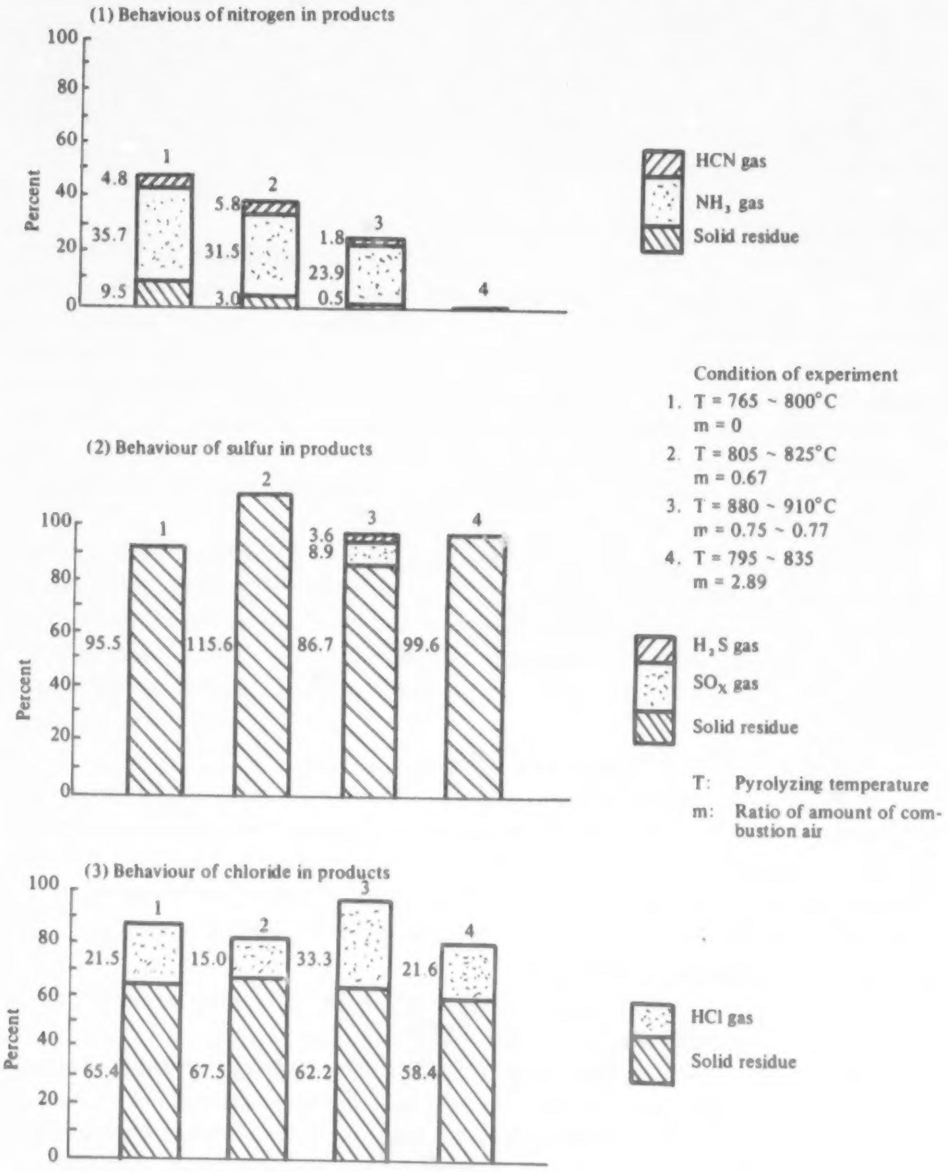
The concentration of dust at the outlet of furnace at the time of pyrolysis was a range between 0.5 ~ 2.87 g/Nm^3, or almost at the same level as the concentration of dust in incineration. However, most of the dust particles generated by pyrolysis are more than 20μ in size. A calculation of the efficiency of dust removal by the scrubber showed that the dust removal rate was a range between 75 ~ 97%.

c. Behavior of Nitrogen in Products

How nitrogen in sludge changes form at the outlet of the furnace as a result of pyrolysis or incineration is shown in Fig. 4.12(1).

In the case of incineration, most of the nitrogen in sludge is estimated to have changed into nitrogen gas. In the case of pyrolysis, about 30% of the nitrogen in sludge exists in the form of ammonia gas, 2 ~ 5% in the form of hydrogen cyanide gas and 1 ~ 10% remains in solid residue, although the amounts differ, depending on operating conditions. The remainder is estimated to turn into nitrogen gas. At the pyrolyzing temperature of 900°C, the amount of hydrogen cyanide gas generated by pyrolysis tended to get larger than at 800°C. These hydrogen cyanide gas and ammonia gas could be decomposed almost completely, when burned in the combustion chamber at the temperature of 1,000°C and at the oxygen concentration of 1 ~ 2%in the chamber.

Fig. 4.12 Bahaviour of Nitrogen, Sulfur and Chloride in Products of Pyrolysis



d. Behavior of Sulfer in Products

How sulfer in sludge changes form by pyrolysis or incineration is shown in Fig. 4.12(2). With the exception of No.3 as shown by the Figure, the sulfer in sludge all remained in solid residue. In the case of incineration (No. 4), too, all the sulfer in sludge remained in solid residue.

Of the data shown in Fig. 4.12(2), No.3 shows an efficient and economical operating condition for pyrolysis, under which part of the sulfer in sludge turns into hydrogen sulfide and SO_x. Since hydrogen sulfide turns into SO_x when burned in the combustion chamber at the temperature of about 1,000°C, it is estimated that, under the operating condition of No. 3, about 12% of the sulfer is considered to change into SO_x.

e. Behavior of Chloride in Products

As shown by Fig. 4.12(3), about 60% of chloride in sludge remains in solid residue, while part of it is converted into hydrogen chloride. But it was not detected in the form of chlorine gas.

Compared with pyrolysis, incineration tends to generate a considerably large amount of hydrogen chloride. In either case, however, the integrated amount of chloride does not reach 100% and it is not known whether there was a measurement error or the remainder exists in the form of another compound.

f. Offensive Odor Components in Exhaust Gas

The results of measurement of offensive odor components in dry exhaust gas is shown in Table 4.6. The measurement was made only on the experiment condition No. 3 (at the pyrolyzing temperature of 880 ~ 910°C with the ratio of amount of combustion air set at 0.75 ~ 0.77). G_1 in the Table shows the sampling point for gas at the outlet of dryer, G_3 at the outlet of pyrolysis furnace and G_4 at the outlet of combustion chamber. Main offensive odor components at G_1 are acetaldehyde and ammonia, and a little amounts of methyl mercaptan, dimethyl disulfide, trimethylamine and carbon disulfide were also detected.

The main offensive odor components at G_3 are hydrogen sulfide, ammonia and formaldehyde. A little amounts of carbon disulfide, trimethylamine and acetaldehyde were also detected.

Exhaust gas sampled at G_4 after gas at G_1 and G_3 are burned and decomposed in the combustion chamber does not contain major offensive odor components at all. Only very small amounts of methyl sulfide, ammonia, acetaldehyde and carbon disulfide are contained in the exhaust gas.

Table 4.6 Analytical Results of Offensive Odor Component in Exhaust Gas

Run No.	Pyrolyzing Temp. (°C)	Ratio of Amount of Combustion Air	Portion of Sampling	H_2S (ppm)	Methyl Mercaptan (ppm)	Methyl Sulfide (ppm)	Tri-Methylamine (ppm)	NH_3 (ppm)	Dimethyl Disulfide (ppm)	Sterane (ppm)	Acetaldehyde (ppm)	Formaldehyde (ppm)	Acetic Acid (ppm)	Carbon Disulfide (ppm)	Degree of Odor (Times)
3	880~ 910	0.75~ 0.77	G1	0.02	0.006	<0.005	0.003	0.09	0.04	<0.02	1.8	<0.05	<2	0.09	100
			G3	11	<0.006	<0.002	0.003	980	<0.001	<0.002	0.002	1.1	<3	0.15	25
			G4	<0.02	<0.006	0.002	0.001	0.74	<0.001	<0.002	0.007	<0.04	<3	0.0005	1

g. Washed Waste Quality of Exhaust Gas and Their Countermeasure

The results of analysis of washed waste of dried exhaust gas are shown in Table 4.7. As shown in the Table, the washed waste contains a little amounts of zinc, copper and iron in addition to 0.002 ~ 0.006 mg/ℓ of mercury. The problem involved in washed waste quality is that of pH. However, it became clear as a result of experiments for practical study.

Table 4.7 Washed Wastewater Quality Results of Exhaust Gas

Run No.	Pyrolyzing Temp. (°C)	Combustion Chamber O_2/CO (Vol %)	pH	BOD (mg/ℓ)	SS (mg/ℓ)	CN^- (mg/ℓ)	Phenol Compound (mg/ℓ)	Hg (mg/ℓ)	As (mg/ℓ)	Cd (mg/ℓ)	Pb (mg/ℓ)	Zn (mg/ℓ)	Cu (mg/ℓ)	Fe (mg/ℓ)	T-Cr (mg/ℓ)	Cr^{+6} (mg/ℓ)
1	1,050~ 1,090	1.4/o	4.3	2	30	<0.01	0.01	0.004	<0.01	<0.01	<0.05	0.22	0.02	7.6	<0.01	<0.01
2	990~ 1,050	1.3/o	4.7	2	16	<0.01	0.02	0.006	<0.01	<0.01	0.09	0.19	0.02	3.2	<0.01	<0.01
3	1,020~ 1090	1.3~ 2.0/o	7.1	2	17	<0.01	<0.03	0.005	<0.01	0.01	0.05	0.26	0.01	3.9	<0.01	<0.01
4	1,060~ 1,100	1.8/o	3.5	2	9	0.03	<0.03	0.002	<0.01	<0.01	<0.05	0.14	0.01	3.7	<0.01	<0.01

4.4 PRACTICAL STUDY ON DRYING-PYROLYSIS PROCESS

Experiment for practical study were conducted on the basis of the results of the basic study mentioned in Section 4.3 by using the pilot plant shown in Fig. 4.2. In the experiments, an indirect steam dryer and a pyrolysis furnace were operated continuously, and the results of the experiments were compiled. The series of experiments were conducted by changing pyrolyzing temperature and feed loading of dewatered cake. The results of the experiments are shown in Table 4.8. Run No. 801 ~ 808 in the Table mean a series of continuous experiments on drying-pyrolysis process, and Run No. 809 was an experiment to feed into the pyrolysis furnace dried cakes stocked in the dry cake feeder. Analysis of products was done only on Run No. 803 and No. 804.

4.4.1 EXPERIMENTS ON DRYING FOR DEWATERED CAKE USING BY INDIRECT STEAM DRYER

The indirect steam drying of dewatered cakes was carried out more efficiently than in basic study. Its Overall Heat Transfer Coefficient (U) was 140 ~ 170 Kcal/$m^2 \cdot hr \cdot °C$, which means a sharp rise in efficiency compared with 70 ~ 110 Kcal/$m^2 \cdot hr \cdot °C$ in the basic study.

The paddle type dryer used in the experiments for practical study was a four-shaft dryer whose mixing frequency is higher than the two-shaft paddle type dryer used in the basic study. It is presumed that, for this reason, the heat transfer efficiency was raised.

4.4.2 EXPERIMENTS ON PYROLYSIS

a. Pyrolyzing Temperature

Fig. 4.13 shows the variations in the ignition loss of solid residue and in the decomposition rate of combustibles, when pyrolyzing temperature was changed between 700 and 900°C. In the experiments, feed loading of dewatered cake was

Table 4.8 Operating Results of Drying – Pyrolysis Process at the Practical Study

Operating Condition	Items / Run No.	801	802	803	804	805	806	807	808	809
Samples of Dewatered Cake	Moisture Contents of Cake (%)	75.8	75.8	75.8	75.8	77.5	77.5	76.5	76.5	(77.5)
	Gross Calorific Value (Kcal/kg·DS)	2900	2900	2900	2900	3140	3140	3090	2960	(3140)
	Ignition Loss of Cake	59.5	59.5	59.5	59.5	61.4	61.4	59.6	59.5	(61.4)
Indirect Steam Dryer	Pressure of Steam Used (kg/cm²·G)	1.6~2.6	2.7~3.8	4.5~5.2	3.6~4.5	1.6~3.4	4.3~5.2	5.1~5.2	5.0~5.3	–
	Temperature of Steam Used (°C)	128~139	139~150	155~159	148~155	130~145	157	150~160	160	–
	Feed Rate of Dewatered Cake (kg/hr.)	154.1	203.2	200.1	182.5	101.4	176.1	239.0	256.0	–
	Moisture Contents of Dried Cake (%)	34.9	36.3	35.1	37.1	24.7	37.1	43.6	40.4	–
	Amount of Dried Cake Discharge (kg/hr.)	57.3	77.2	74.6	71.1	30.3	63.0	99.6	101.0	–
	Amount of Vaporized Water (kg-H_2O/hr.)	96.8	126	125.5	113.7	71.1	113.1	139.4	155.2	–
	Amount of Condensed Water (kg/hr.)	135	158	174	170	126	185.1	183	192	–
	Shaft and Screw Side (kg/hr.)	82	98	105	103	75	113.9	117	119	–
	Jacket Side (kg/hr.)	53	60	69	67	51	72.2	66	73	–
	Amount of Heat Transfer [Input] (Kcal/hr.)	67,635	78,327	85,532	83,881	63,000	90,884	89,712	93,982	–
	Overall Heat Transfer Coef. (Kcal/m²·hr·°C)	176.2	168.6	142.8	156.7	155.9	151.2	146.6	150.1	–
Combustion Chamber	Temperature of Combustion Chamber (°C)	1100~1170	1030~1120	1000~1090	1035~1130	960~1055	1070~1130	1080~1150	1090~1160	960~1000
	Fuel Consumption (ℓ/hr.)	10.3	8.6	4.8	5.9	5.2	8.3	10.2	9.9	4.0
	CO/O_2	0/0.4~1.5	0/0.9~3.0	0/1.2	0/1.3	0/1.2	0.1/0.8~1.3	0/1.2	0/0.5~1.2	0/2.1
Exhaust Gas Boiler	Outlet Temp. of Exhaust Gas Boiler (°C)	204~210	208~226	230~240	219~235	190~198	215~225	216~245	238~245	–
	Pressure of Boiler (kg/cm²·G)	3.6~4.1	4.3~5.3	6.0~6.5	5.4~6.0	3.2~4.5	5.8~6.8	5.2~7.0	7.2~7.6	–
	Amount of Steam Generated (kg/hr.)	204.3	211.2	212.7	212.8	210.8	212.5	213.4	216.7	–
	Amount of Steam Discharged (kg/hr.)	0	0	0	0	0~20	0	0	0	–
No.1 Scrubber and Dehumidifier	Flow-rate of Scrubbing Water (m³/hr.)	3.6~3.9	2.0~2.1	3.8~3.9	2.8~3.2	0.9~2.3	1.2	3.6~3.9	3.4~3.5	–
	Temperature at Outlet of Scrubber (°C)	24~30	28~30	28~32	27~30	21~31	20~30	32~35	33~36	–
	Amount of Exhaust Gas (m³/hr.)	35~120	50~75			50~113		65~95	40~70	–
No.2 Scrubber and Dehumidifier	Flow-rate of Scrubbing Water (m³/hr.)	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
	Temperature at Outlet of Scrubber (°C)	24~27	45~48	42~50	46~52	32~55	42~52	41~59	33~36	47~52
Pyrolysis Furnace	Feed Rate of Dried Cake (kg/hr.)	57.3	77.2	74.6	71.1	30.3	63.0	99.6	101.0	103.9
	Moisture Contents of Dried Cake (%)	34.9	36.3	35.1	37.1	24.7	37.1	43.6	40.4	28.8
	Feed Loading of Dried Cake (kg·DS/m²·hr.)	23.0	30.4	29.9	27.6	14.1	24.4	34.7	37.2	43.8
	Amount of Combustion Air Supply (Nm³/hr.)	83	80	86	86	48.5	92	108	128	148
	Distribution Rate, 1:2:3:4	0:39:44:0	0:36:44:0	0:42:44:0	0:42:44:0	0:20:28.5:0	0:46:46:0	0:54:54:0	0:64:64:0	0:74:74:0
	Ratio of Amount of Combustion Air	0.77	0.56	0.61	0.66	0.68	0.74	0.62	0.72	0.66
	Amount of Propane Gas Consumed	0.3	0.5	0.7	0.7	1.4	1.0	0.9	0.5	0.3
	Temperature 1st Hearth	465~575	770~795	810~885	870~905	890~930	870~920	795~825	805~895	885~920
	2nd Hearth	695~720	775~840	890~920	875~920	900~930	885~930	880~915	890~920	890~930
	3rd Hearth	775~810	815~830	910~935	880~905	875~930	880~920	920~940	910~955	880~905
	4th Hearth	705~785	790~830	880~920	890~920	860~900	900~930	870~890	880~920	810~925
	Detention Time of Dried Cake (min.)	60	60	60	60	60	60	60	60	60
	Static Pressure (mmAq)	+3~+20	+0~+1.2	+7~+15	+15~+25	+7~+20	+2~+10	+10~+20	+10~+20	+3~+20
	Outlet Gas CO, H_2, CH_4 Vol. (%) Vol. (%)	0.57, 0.12, 0.04	1.72, 0.99, 0.42	1.34, 1.64, 0.34	0.95, 1.35, 0.24	1.40, 1.62, 0.34	0.15, 0.37, ND	1.72, 1.90, 0.16	0.57, 0.21, ND	1.11, 1.73, 0.24
	C_2H_2, C_3H_6, O_2 Vol. (%) Vol. (%)	ND, ND, 0.31	0.19, ND, 0.27	0.11, ND, 0.17	0.12, ND, 0.26	ND, ND, 0.13	ND, ND, 0.53	1.04, ND, 0.20	ND, ND, 0.2	ND, ND, 0.1
	Calorific Value (Kcal/Nm³)	24.8	181.2	139.6	110.7	124.3	21.9	131.4	23.7	97.1
Heat Source Chamber	Circulating Gas Temperature (°)	202~210	208~226	230~240	219~235	190~198	215~226	216~245	238~245	186~204
	Flow Rate of Circulating Gas (m³/hr.)	40	72.4	41.2	42.2	50	45.1	32.3	26.8	53.6
	Amount of Fuel Consumed (ℓ/hr.)	3.5	6.6	8.1	7.3	7.3	6.5	7.7	7.0	6.7
	Amount of Air Supplied (Nm³/hr.)	44	75.2	73.5	65.8	62.0	58.1	54.3	55.0	59.2
	Temperature in Heat Source Chamber (°C)	760~890	860~940	1030~1100	1050~1130	1025~1075	920~990	1000~1100	1085~1190	940~1040
Quality of Solid Residue	Amount of Solid Residue (Dry·kg/hr.)	18.9	23.5	21.3	19.8	8.9	15.8	24.7	26.8	31.2
	Ignition Loss (%)	20.0	15.2	8.1	8.5	0.9	3.3	7.9	9.3	12.3
	Decomposition Rate of Combustible (%)	83.0	87.8	94.0	93.7	99.4	97.9	94.2	93.0	91.2
	Solved Concentration of Cr^{+6} (mg/l)	ND	ND	ND	ND	ND	ND	ND	ND	ND
	Cr^{+6} Contents (mg/kg)	3.5	4.8	5.8	6.0	3.6	2.4	0.9	0.9	1.7
Fuel Consumption	(l/hr.)	14.5	16.5	14.7	14.8	16.1	17.4	20.3	18.2	–
	(l/ton-Cake)	94.2	81.2	73.7	81.3	159.4	78.7	84.9	71.2	–

Remark: Run No. 809 is Experiment of Pyrolysis Process by Dried Cake Feed.

set at about 30 kg·DS/m²·hr and the ratio of amount of combustion air at about 0.6. As shown by the Figure, it was found that, if pyrolysis is done at temperatures of 900°C or more, the ignition loss of solid residue is below 10%.

And Fig. 4.14 shows the relationship between pyrolyzing temperatures and fuel consumption. Given in the Figure are the fuel consumption in the pyrolysis furnace alone and the total fuel consumption in the pyrolysis furnace and the combustion chamber. The amount of fuel consumed in the pyrolysis furnace can be divided into fuel consumed in the heat source chamber and fuel directly supplied in the pyrolysis furnace. Propane gas burners were used for direct heating of the pyrolysis furnace. Listed in the Figure is the amount of fuel which was converted, on the basis of calorific value, from the amount of propane gas used in the pyrolysis furnace for direct heating.

Fig. 4:13 Variation of Both Ignition Loss of Solid Residue and Decomposition Rate in Combustible to Pyrolyzing Temperature

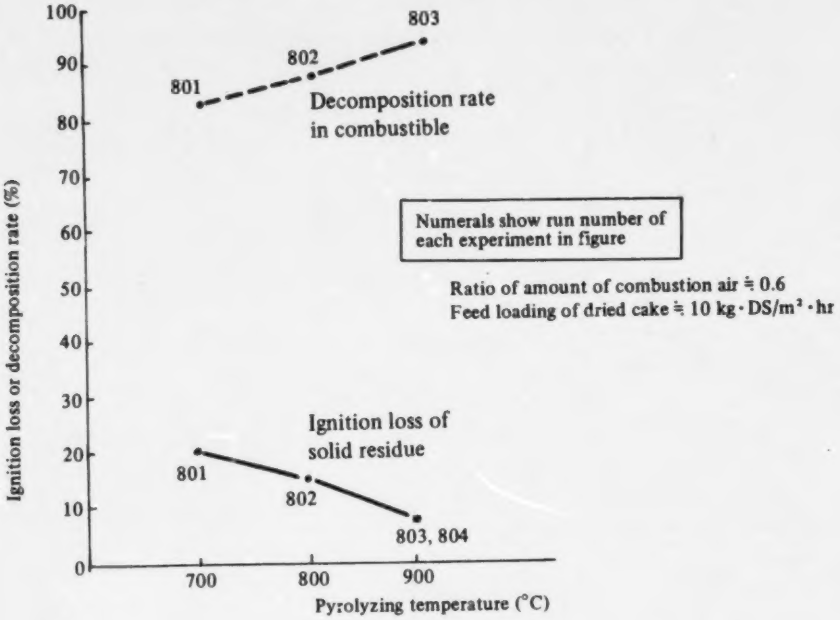


Fig. 4.14 Relationship between Pyrolyzing Temperature and Fuel Consumption

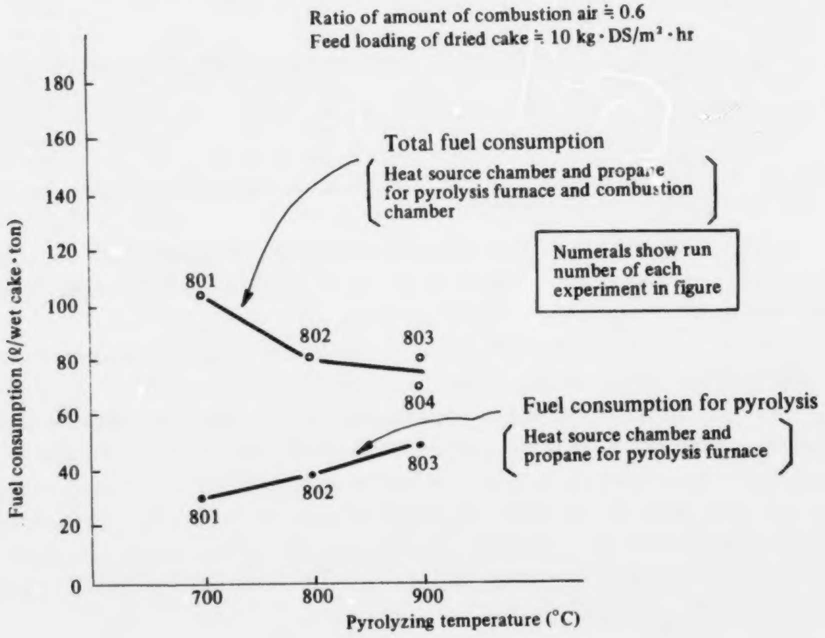


Fig. 4.14 shows that, if operation of pyrolysis is carried out at low temperatures, the amount of fuel supplied to the pyrolysis furnace is reduced, but the total amount of fuel used in the furnace and the combustion chamber tends to increase.

Considering from these results of experiments, it might be said that effective temperature of the dried cake pyrolysis is at about 900°C.

b. Feed Loading of Dewatered Cake.

In the experiments for practical study, the feed loading of dewatered cake was changed in the range of 14 ~ 44 kg·DS/m^2·hr, while the pyrolyzing temperature was kept at about 900°C and the ratio of amount of combustion air at about 0.6. Fig. 4.15 shows the relationship between feed loading of dewatered cake, ignition loss of solid residue and the decomposition rate of combustibles. When the feed loading of dewatered cake was below 25 kg/m^2·hr, as shown in the Figure, the ignition loss of solid residue could be decreased almost to the same extent as in the case of incineration. If the ignition loss is allowed to be 10%, the maximum feed loading of dewatered cake, in case the water content of the cake is around 30% at the inlet of the pyrolysis furnace, can be increased to 40 kg·DS/m^2·hr. It means an increase of 60% over the 25 kg·DS/m^2·hr, the feed loading of dewatered cake obtained in the basic study.

Fig. 4.15 Variation of Both Ignition Loss of Solid Residue and Decomposition Rate in Combustible to Feed Loading of Dried Cake

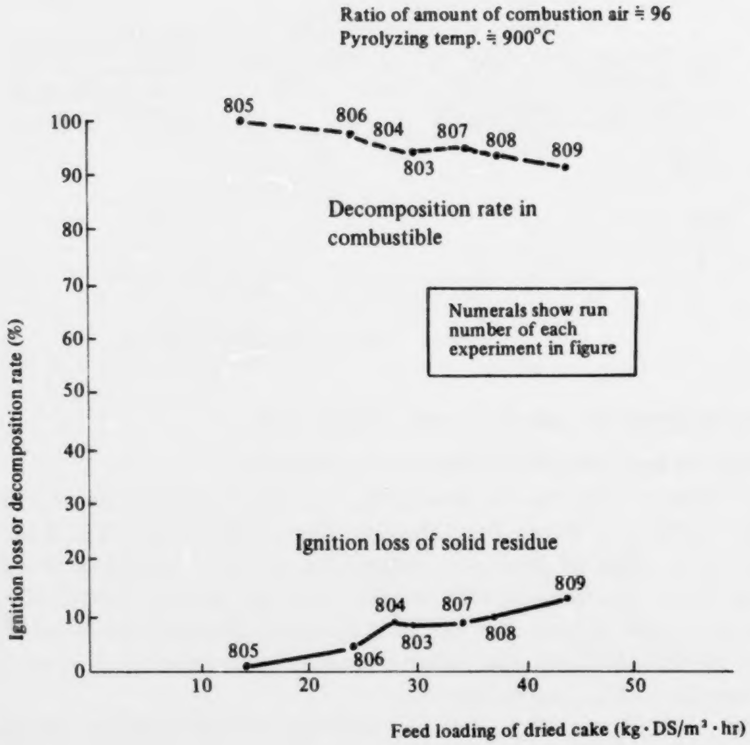
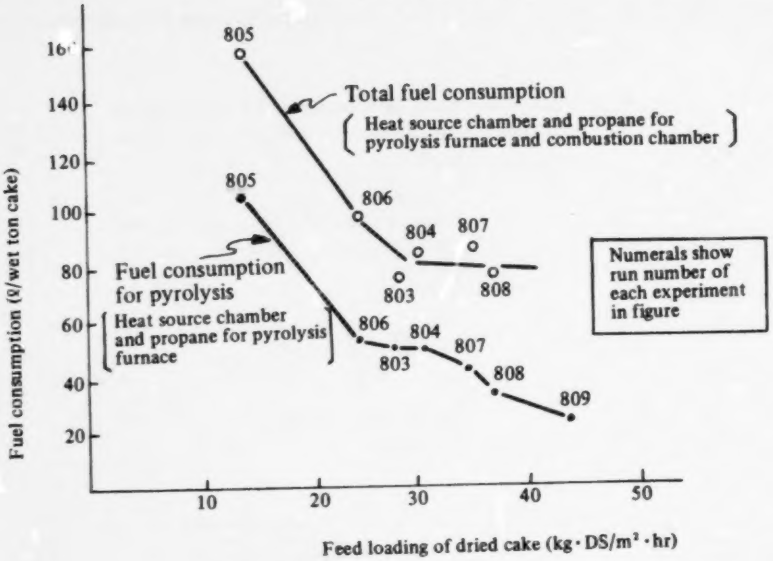


Fig. 4.16 shows the relationship between feed loading of dewatered cake and fuel consumption. The classification of the fuel consumed was made in the same method as in the foregoing Fig. 4.14. It is learned that fuel consumption is greatly influenced by feed loading of dewatered cake into the pyrolysis furnace. When feed loading of dewatered cake is small, fuel consumption increases. In case the feed loading of dewatered cake is larger than 25 kg·DS/m^2·hr, fuel consumption is 80 ~ 90 litre per ton·wet cake.

In case the allowable ignition loss of solid residue is set at 10% as mentioned earlier, and when the feed loading of dewatered cake is 40 kg·DS/m^2·hr, the amount of fuel used for pyrolysis is about 30 litre per ton·wet cake and the total fuel consumption is about 80 litre per ton·wet cake.

Fig. 4.16 Variation of Fuel Consumption to Feed Loading of Dried Cake

Ratio of amount of combustion air ≒ 0.6
Pyrolyzing temp. ≒ 900°C



4.4.3 PRODUCTS OF DRIED CAKE PYROLYSIS

a. Remaining Rate of Each Element in Solid Residue

The results of analyses of dewatered cake and solid residue are shown in Table 4.9. Table 4.10 shows the remaining rate of each element in solid residue calculated on the basis of Table 4.9. From Table 4.9 and Table 4.10, the elements other than those remaining in solid residue show the amount or rate of elements which are estimated to have gone out of the pyrolysis furnace. (Since exhaust gas is treated in the scrubber and combustion chamber, the amount of elements actually emitted into the air is decreased sharply.)

In the measurement of the amount remaining in solid residue or the remaining rate, results different from those in the basic study were obtained regarding some elements. The elements found to have a high remaining rate in the experiments for practical study was chloride, whose remaining rate was about 20% higher than in the basic study. In contrast, the remaining rate of sulfer was about 30% lower than in the basic study. In other words, most of the sulfer remained in solid residue in the basic study, while the remaining rate of sulfer was below 60% in the experiments for practical study. With regard to heavy metals, the amounts of cadmium and lead remaining in solid residue decreased. It is considered that these changes are related to pyrolyzing temperature.

The remaining rate of each element in solid residue changes according to pyrolyzing temperature and it does not follow that pyrolysis is particularly superior to incineration.

However, shown in Table 4.9 is an example denoting the superiority of pyrolysis over incineration. It concerns the fact that part of the hexavalent chromium compounds in sludge is reduced by pyrolysis. This means that, unlike incineration, pyrolysis does not form any new hexavalent chromium compounds but can reduce even part of the compounds contained in sludge. This is a point where pyrolysis is extremely superior to incineration.

Table 4.9 Analytical Results of Both Dewatered Cake and Solid Residue

Items	Run No.	Ignition Loss (%)	Gross Calorific Value (Kcal/kg·DS)	C (%)	H (%)	N (%)	S (%)	O (%)	Cl (%)	CN (ppm)	NH_3 (ppm)
Dewatered Cake	803	59.5	2,960	28.4	4.1	3.1	1.1	27.0	0.65	6.9	2,900
	804	59.5	2,860	28.3	4.2	3.1	1.0	27.0	0.63	3.5	1,400
Solid Residue	803	8.1	320	3.0	0.18	<0.5	1.5	13.0	1.26	0.23	910
	804	8.5	340	3.9	0.18	<0.5	1.3	13.0	1.26	0.20	170

Items	Run No.	Hg (ppm)	As (ppm)	Cd (ppm)	Pb (ppm)	Zn (ppm)	Cu (ppm)	Fe (ppm)	T-Cr (ppm)	Cr^{+6} (ppm)	Cr^{+6}/T-Cr (%)
Dewatered Cake	803	0.56	5.4	11	270	1,200	640	32,600	330	8.6	2.6
	804	0.54	5.0	10	240	1,300	610	31,500	290	7.8	2.6
Solid Residue	803	<0.05	6.9	3.4	98	2,500	1,500	75,200	590	5.9	1.0
	804	<0.05	8.1	3.6	100	2,900	1,500	75,800	730	6.0	0.8

Table 4.10 Remaining Rate of Each Element at the Cake of Drying – Pyrolysis Process

Run No.	Gross Calorific Value (%)	C (%)	H (%)	N (%)	S (%)	O (%)	CL (%)	CN (%)	NH_3 (%)	Hg (%)	As (%)	Cd (%)	Pb (%)	Zn (%)	Cu (%)	Fe (%)	T-Cr (%)
803	4.8	4.6	1.9	<7.1	59.9	4.8	85.4	1.5	13.8	<1.5	56.3	13.6	16.0	91.8	103.3	101.7	78.8
804	5.3	6.2	1.9	<7.2	57.5	5.3	88.5	2.5	5.4	<1.5	71.7	15.9	18.4	98.7	108.8	106.5	111.4

b. Concentration and Particle Size Distribution of Dust

As shown by Table 4.11, the concentration of dust from the pyrolysis furnace is considerably high. The concentration of dust is lowered sharply after dust is sent through the scrubber, but it is necessary to install an electric precipitator or other equipment in order to hold down the dust concentration rate below 0.34 g/Nm^3.

Much of the dust from the pyrolysis furnace has a relatively large particle size as shown in Table 4.12. Accordingly, it is considered to be comparatively easy to separate dust by an electric precipitator or other equipment.

Table 4.11 Characteristics of Exhaust Gas (Run No.803)

Sampling Site	Temp. of Gas (°C)	Velocity of Gas (m/sec.)	Moisture Contents (Vol. %)		Volume of Dry Gas (Nm^3/hr.)		Dust (g/Nm^3)	HCN (ppm)	NH_3 (ppm)	NOx (ppm)	SOx (ppm)	HCl (ppm)	Cl_2 (ppm)	Degree of Odor (Times)
			Actual	Calculation	Actual	Calculation								
G_2	–	–	–	14.5		103	–	–	–	170	130	–	<0.2	–
G_3	760	13.8	24.6	24.7	200	213	8.4	240	2.080	54	130	6.0	<0.2	400
G_4	215	16.1	20.8	21.8	330	308	3.1	<0.1	1.0	210	230	15.0	<0.2	2
G_5	49	1.4	10.3	10.4	280	275	0.34	<0.1	0.5	190	130	1.4	<0.2	2

Remark: G_2 : Exhaust Gas Samples from Heat Source Chamber
G_3 : Exhaust Gas Samples from Pyrolysis Furnace
G_4 : Exhaust Gas Samples from Exhaust Gas Boiler
G_5 : Exhaust Gas Samples from Gas Scrubber

Table 4.12 Dust Size Distribution in Exhaust Gas from Pyrolysis Furnace (Run No.804)

(%)

Size (μ) / Samples	>6.5	4.3~6.5	2.8~4.3	1.9~2.8	1.2~1.9	0.6~1.2	0.4~1.6	0.2~0.4	<0.2
1	73.8	12.1	6.1	3.2	1.5	0.6	0.9	1.2	0.6
2	70.2	12.9	5.8	3.6	2.1	0.9	1.0	1.3	2.2

c. Components of Exhaust Gas

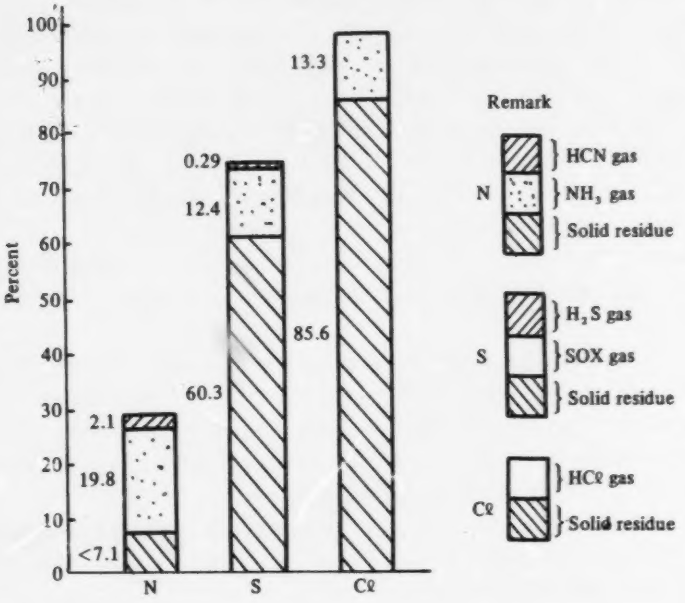
Table 4.11 shows the results of analyses of such exhaust gas components as hydrogen cyanide, ammonia, NO_X and SO_X. The amounts of various exhaust gas components generated in the pyrolysis furnace diminish sharply in the process of decomposition in the combustion chamber and cleaning by the scrubber. For this reason, hydrogen cyanide, ammonia and hydrogen chloride will do no harm. However, the amounts of NO_X and SO_X do not decrease. Especially, the concentration of NO_X is much higher in exhaust gas than at the outlet of the pyrolysis furnace, since it is treated in the combustion chamber at temperatures over 1,000°C. Thus it may become necessary to take steps to remove SO_X and NO_X, as in the case of incineration, where the environmental quality standard on air pollution is being enforced strictly.

d. Behavior of Nitrogen, Sulfer and Chloride in Sludge

The forms of nitrogen, sulfer and chloride at the outlet of the pyrolysis furnace is shown in Fig. 4.17. There is hardly any difference in the results of analysis of nitrogen between the experiments for practical study and the basic study (See Fig. 4.12). In the case of sulfer, the results of analysis differed from those in the basic study, with the whereabout of about 27% of the substance unknown. As a result of measurement, it was found that the amount of sulfer remaining in solid residue decreased, while an increased amount of SO_X was generated.

The amount of chloride remaining in solid residue tended to go up by about 20% from the level recorded in the basic study.

Fig. 4.17 Forms of Each Element in Product of Pyrolysis



e. Offensive Odor Components in Exhaust Gas

Table 4.13 shows the results of analyses of offensive odor components in exhaust gas. Obtained from the analyses were numerical values almost similar to those in the basic study. The main offensive odor components generated in the indirect steam dryer are ammonia and acetaldehyde, and those generated in the pyrolysis furnace are high concentrations of ammonia, hydrogen sulfide and formaldehyde.

Although most of the components are decreased in amount through the decomposition in the combustion chamber, the amounts of ammonia, acetaldehyde and formaldehyde remaining in exhaust gas were found to be considerably higher than those recorded in the basic study.

Table 4.13 Analytical Results of Offensive Odor Component in Exhaust Gas

Sampling Site	H_2S (ppm)	Methyl Mercaptan (ppm)	Methyl Sulfide (ppm)	Tri-methyl Amine (ppm)	Ammonia (ppm)	Dimethyl Disulfide (ppm)	Sterane (ppm)	Acetaldehyde (ppm)	Formaldehyde (ppm)	Acetic Acid (ppm)	Carbon Disulfide (ppm)
G_1	0.07	<0.005	0.08	0.005	8.7	0.5	<0.3	1.0	<0.15	<3.0	0.07
G_3	5.0	<0.005	<0.003	0.002	2.080	0.03	<3	0.1	3.9	<3.0	0.6
G_4	<0.008	<0.005	<0.003	0.002	1.0	<0.002	<0.02	0.07	0.44	<3.0	0.006

Remark: G_1 : Exhaust Gas Sample from Indirect Steam Dryer
G_3 : Exhaust Gas Sample from Pyrolysis Furnace
G_4 : Exhaust Gas Sample from Exhaust Gas Boiler

4.4.4 COMPARISON OF DRYING-PYROLYSIS PROCESS, DIRECT PYROLYSIS PROCESS AND INCINERATION PROCESS

Experiments were conducted on the direct pyrolysis process and incineration process of dewatered cakes to find out the merits and demerits of the drying-pyrolysis process. The results of the experiments are shown in Table 4.14. It shows the results of the Run No. 808 as an example of the drying-pyrolysis process. The properties of the dewatered cake used in Run No. 808 in Table 4.14 is a little different from the dewatered cakes used in Run No. 903 and No. 904. The cake used in Run No. 808 has less combustibles than dewatered cakes used in Run No. 903 and No. 904 and, therefore, its calorific value is small. It was difficult to compare the three processes exactly, but it was concluded that a tentative comparison would be possible.

In Table 4.14, Column I shows empirical values and Column II the values revised for practical reasons. The revision was made in the following manners.

i) In the drying-pyrolysis process, it was assumed that the temperature of returning water from the drain was 80°C and that measures were taken to prevent white smokes of exhaust gas from gonig out of the chimney.

ii) In the direct pyrolysis process and incineration process of dewatered cakes, it was assumed that heat exchange was done by a heat exchanger, in addition to the revision made in above Article i).

Under these revisions, fuel consumption for the drying-pyrolysis process rises higher than the empirical value and fuel consumption for direct pyrolysis process and incineration process goes down from the empirical values. Nevertheless, fuel consumption for the drying-pyrolysis process is about 1/2 of that needed for direct pyrolysis process and about 1/3 of that for incineration process.

The amount of exhaust gas generated by drying-pyrolysis process becomes considerably smaller than that generated by direct pyrolysis process or incineration process. This means that the drying-pyrolysis process could lower the costs of construction and operation of facilities to prevent air pollution, by a larger margin than direct pyrolysis process and incineration process.

Comparing direct pyrolysis process with incineration process, the former is also considered to be considerably more advantageous than the latter economically.

The experiments were conducted at a pilot plant, and the operation of a practical plant is expected to produce results considerably different from those obtained in the foregoing experiments. For example, in the experiments of Run

Table 4.14 Comparable Experimental Data of Drying-Pyrolysis Process, Direct Pyrolysis Process and Incineration Process

Items		Process	Drying-Pyrolysis	Direct Pyrolysis	Incineration
		Run No.	808	903	904
Moisture Contents of Dewatered Cake (%)			76.5	77.8	77.5
Ignition Loss of Cake (%)			59.5	66.0	66.0
Ash Contents of Cake (%)			40.5	34	34
Gross Calorific Value (Kcal/kg·DS)			2,900	3,300	3,300
Moisture Contents at Inlet of Pyrolysis Furnace (%)			40.4	77.8	77.5
Ratio of Amount of Combustion Air			0.71	0.60	2.0
Exhaust Gas Temperature at Outlet of Pyrolysis Furnace (°C)			850	393	485
Exhaust Gas Temperature at Outlet of Combustion Chamber (°C)			1,100	800	800
Fuel Consumption (l/ton·Cake)	I	Furnace	33	144	128
		Combustion Chamber	39	89	234
		Total	72	233	362
	II	Furnace	33	144	128
		Combustion Chamber	61	44	162
		Total	94	188	290
Dry Exhaust Gas (Nm^3/ton·Cake)	I	Furnace	858	1,495	2,673
		Combustion Chamber	1,428	2,227	4,968
	II	Furnace	858	1,495	2,673
		Combustion Chamber	1,556	2,184	4,425
Wet Exhaust Gas (Nm^3/ton·Cake)	I	Furnace	1,199	2,748	3,949
		Combustion Chamber	1,651	2,381	5,350
	II	Furnace	1,199	2,748	3,949
		Combustion Chamber	1,651	2,339	4,727

No. 903 and No. 904, the temperatures at the outlet of the furnace were considerably higher than at the outlet of the furnace for practical use.

For this reason, trial calculations were made on drying-pyrolysis, direct pyrolysis and incineration in a practical use furnace (Wet Base 50 t/day) by introducing the idea of designing a conventional multi-hearth furnace. The results of the calculations are shown in Table 4.15.

In the drying-pyrolysis process, as shown in the Table, no fuel is supplied to the pyrolysis furnace at all since sludge cake is in a dry state. And only a small amount of fuel is needed for the process because exhaust gas generated in the pyrolysis furnace is used as part of the heat source for boiler. In direct pyrolysis, fuel consumption for the pyrolysis furnace is the largest, but that for the combustion chamber is less than for the drying-pyrolysis process because no fuel is needed for the boiler in the combustion chamber. In incineration, less fuel is supplied to the furnace than in direct pyrolysis since combustibles are burned in the furnace, but the amount of fuel used in the combustion chamber for the combustion of exhaust gas is very large. But if there is no combustion of exhaust gas, the amount of fuel for incineration is the smallest. In reality, however, sewage treatment plants in most of the municipalities in Japan are equipped with facilities for

Table 4.15 Comparable Calculation Results of Drying-Pyrolysis Process, Direct Pyrolysis Process and Incineration Process

Process Items		Drying-Pyrolysis Process	Direct Pyrolysis Process	Incineration Process
Moisture Contents of Dewatered Cake (%)		78	78	78
Ignition Loss of Cake (%)		66.0	66.0	66.0
Ash Contents of Cake (%)		34	34	34
Gross Calorific Value (Kcal/kg·DS)		3,300	3,300	3,300
Moisture Contents at Inlet of Pyrolysis Furnace (%)		25	78	78
Ratio of Amount of Combustion Air		0.60	0.60	2.0
Exhaust Gas Temperature at Outlet of Pyrolysis Furnace (°C)		700	200	200
Exhaust Gas Temperature at Outlet of Combustion Chamber (°C)		1,100	800	800
Fuel Consumption (l/ton·Cake)	Furnace	0	44	27
	Combustion Chamber	52	33	120
	Total	52	77	147
Dry Exhaust Gas (Nm^3/ton·Cake)	Furnace	338	879	2,023
	Combustion Chamber	1,302	1,467	3,302
Wet Exhaust Gas (Nm^3/ton·Cake)	Furnace	542	1,985	3,172
	Combustion Chamber	1,408	1,636	3,638

the combustion of exhaust gas or for the deodorization by catalyzer in order to prevent the generation of offensive odor and the emission of white smokes from chimneys.

Pyrolysis process can decrease exhaust gas sharply, compared with incineration process. Therefore, pyrolysis process requires less costs of construction and operation of such exhaust gas control facilities as electric precipitators and alkali absorption scrubber. In this respect, pyrolysis process can be said to have a great economic advantage.

4.5 SUMMARY

Basic and practical studies were carried out on the drying-pyrolysis process in order to put to practical use the pyrolysis of sewage sludge by a multi-hearth furnace. The sampled sludge for use in these studies was dewatered cake of raw sludge at the Toba Sewage Treatment Plant in Kyoto. The pilot plant for the basic study was made up of a two-shaft indirect steam dryer, a single-hearth furnace (with a total furnace floor space of 2.1 m^2) and its incidental facilities. The pilot plant for the experiments for practical study consisted of a four-shaft indirect steam dryer, a four-hearth furnace (with a total furnace floor space of 1.63 m^2) and its incidental facilities.

The results of the studies can be summarized as follows.

(1) As a result of the experiments for practical study, the Overall Heat Transfer Coefficient (U) of the indirect steam dryer for sludge cakes drying could be raised to 140 ~ 170 $Kcal/m^2 \cdot hr \cdot {}^\circ C$. It is larger than the Overall Heat Transfer Coefficient of 70 ~ 110 $Kcal/m^2 \cdot hr \cdot {}^\circ C$ obtained in the basic study. This is ascribed to the increase in the number of shafts of the indirect steam dryer.

(2) For the drying-pyrolysis process, it is appropriate to set the ratio of amount of combustion air at 0.5 ~ 0.7 and pyrolyzing temperature at 900°C in order to keep the ignition loss of solid residue below 10%. According to the experiments for practical study, adequate feed loading of dewatered cake to maintain the same condition is estimated to be 40 $kg \cdot DS/m^2 \cdot hr$.

(3) Pyrolysis can completely prevent the oxidization into hexavalent chromium compounds, of trivalent chromium compounds contained in sludge cakes. And part of the hexavalent chromium compounds contained in sludge cake are reduced in the furnace by reducing gas generated in the pyrolysis furnace.

(4) At the basic and practical studies, there were found to produce slightly different results with regard to the remaining rate of each element in solid residue from pyrolysis. The difference relates to pyrolyzing temperature. The remaining rate of each element in solid residue is related to temperature, and it does not follow that pyrolysis is by far superior to incineration.

(5) The amount of dust generated in pyrolysis furnace is considerably large. Furthermore, many particles are large in size. Although most of the dust can be removed when passed through the scrubber, it is also necessary to install an electric precipitator for maintaining the concentration of dust at a very low level as same as the case of incineration process.

(6) Of the exhaust gas components generated in the pyrolysis furnace, hydrogen cyanide and ammonia are decomposed in the combustion chamber. Hydrogen chloride can be removed by means of a gas scrubber. Since SO_x and NO_x cannot be removed, however, it may become necessary to take measures to deal with the problem in areas where the environmental quality standard on air pollution is strictly enforced.

(7) The main offensive odor components in exhaust gas generated in the indirect steam dryer are ammonia and acetaldehyde, and those generated in the pyrolysis furnace are much amount of ammonia, hydrogen sulfide and formaldehyde. Most of these components are diminished through the decomposition in the combustion chamber, but small amounts of ammonia, acetaldehyde and formaldehyde remain in exhaust gas at the outlet of combustion chamber.

(8) The drying-pyrolysis process consumes the least amount of fuel in case there is combustion of exhaust gas. But, in case the combustion of exhaust gas is not needed, incineration process consumes the least amount of fuel.

The drying-pyrolysis process and direct pyrolysis process can reduce the generation of exhaust gas to below 1/2 of the amount of exhaust gas generated by incineration process. Therefore, pyrolysis process is expected to have a great economic efficiency in case steps are necessary to prevent air pollution and offensive odor.

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(1) T. Majima et al, Studies on Pyrolysis Process of Sewage Sludge, Proceedings of the 8th International Conference on Water Pollution Research, Sydney, p. 381 (1976)

FIFTH US/JAPAN CONFERENCE
ON
SEWAGE TREATMENT TECHNOLOGY
PAPER NO. 4

DEVELOPMENT AND EVALUATION OF AUTOMATIC WATER QUALITY MONITORING EQUIPMENT

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JAPANESE GOVERNMENT

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DEVELOPMENT AND EVALUATION OF AUTOMATIC WATER QUALITY MONITORING EQUIPMENT

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DEVELOPMENT AND EVALUATION OF AUTOMATIC WATER QUALITY MONITORING EQUIPMENT

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DEVELOPMENT AND EVALUATION OF AUTOMATIC WATER QUALITY MONITORING EQUIPMENT

1. FOREWORD

In the field of sewage works, there is a pressing need for the development of reliable automatic water quality measuring devices for monitoring sewage quality including industrial wastewater discharged into the public sewage system and for automatic control of treatment processes.

The Ministry of Construction gives high priority to the development of measuring devices for detecting toxic substances that may be contained in industrial wastewater discharged into the public sewage system. The development of such measuring devices are now being carried out by a contract research with the Association of Electrical Engineering.

The Electrical Engineering Association organized a committee composed of representatives from nine manufacturers and users for this purpose, and has been conducting field tests on the previously developed devices to seek for any possible technical improvement and developing new instruments with the cooperation of the manufacturers.

The development of measuring instruments for automatic control of treatment processes has also progressed considerably these days. Although their application to automatic control is not so popular yet, these instruments are being installed in many sewage treatment plants for manual control or monitoring.

This report summarizes the activities in the field of research and development of water quality measuring devices in Japan.

2. AUTOMATIC MEASURING DEVICES TO MONITOR QUALITY OF RAW SEWAGE

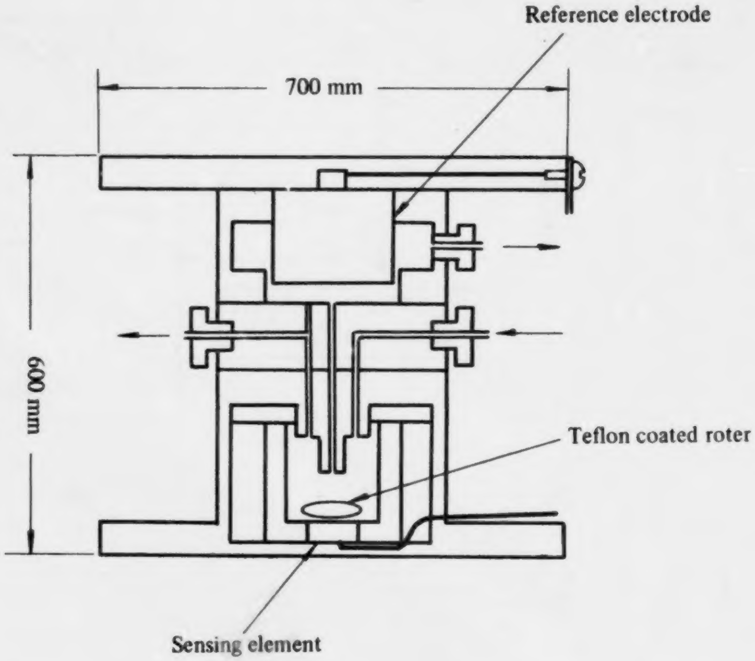
2.1 TOTAL CYANIDE MONITOR

On a contract research basis entrusted to the Association of Electrical Engineering, work has begun to develop a total cyanide monitor from FY 1975. At that time, five models of automatic cyanide monitors from different manufacturers were on the market. Those units, however, were all designed for monitoring cyanides in industrial wastewater which is usually easy to deal with, and the applicability of these products to sewage was unclear. Therefore, field tests of these units were conducted at a sewage treatment plant for about three months. It was revealed that the two models were completely uncapable of continuous operation and the other models, although giving comparatively good results, all had defects. Therefore, efforts were made to develop the New Standard Models by improving previous models. Three prototype models were assembled.

Models I and II are batch-type measuring instruments. Model I incorporats the method of removing sulfides during distillation while Model II employs the method of removing sulfides after distillation. Type III, on the other hand, is of a continuous measuring type.

All of these prototype models utilize cyanide electrodes as the detector unit. Models I and II have two kinds of electrodes: conventional type electrode with an abrasive unit, and an electrode with the measuring cell itself being a sensing element as shown in Fig. 1.

Fig. 1 New Type Cyanide Electrode



The surface of the sensing element of the latter type electrode is constantly cleaned by a rotor. This electrode, when compared with conventional ones, has a lower detecting limit giving a longarithmicly linear output down to 0.01 mg/ℓ.

Fig. 2 shows the flow diagram of Model I monitor. After measuring 100 mℓ of sulfuric acid (1 + 9) and 10 mℓ of N/10 potassium permenganate solution is added to oxidize sulfides and other interferences. Next, before raising the temperature too high, 10 mℓ of N/10 sodium oxalate is added to remove excessive potassium permenganate without decomposing cyanides. The distilled cyanide is absorbed in a 0.4% sodium hydrate solution and measured by cyanide electrodes. Measurements are done at 30 minute intervals.

Fig. 2 Flow Diagram of Total Cyanide Monitor, Model I

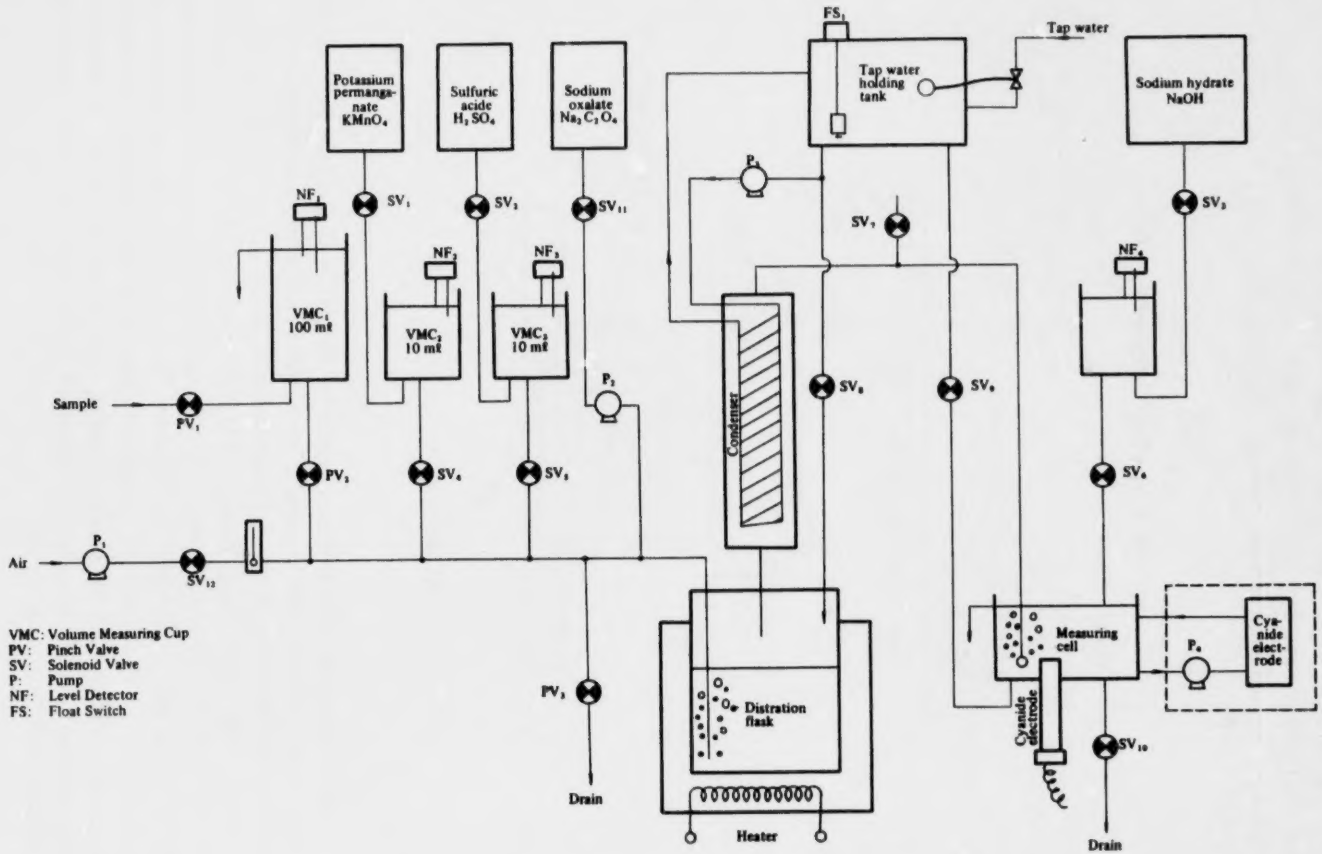


Fig. 3 Flow Diagram of Total Cyanide Monitor, Model II

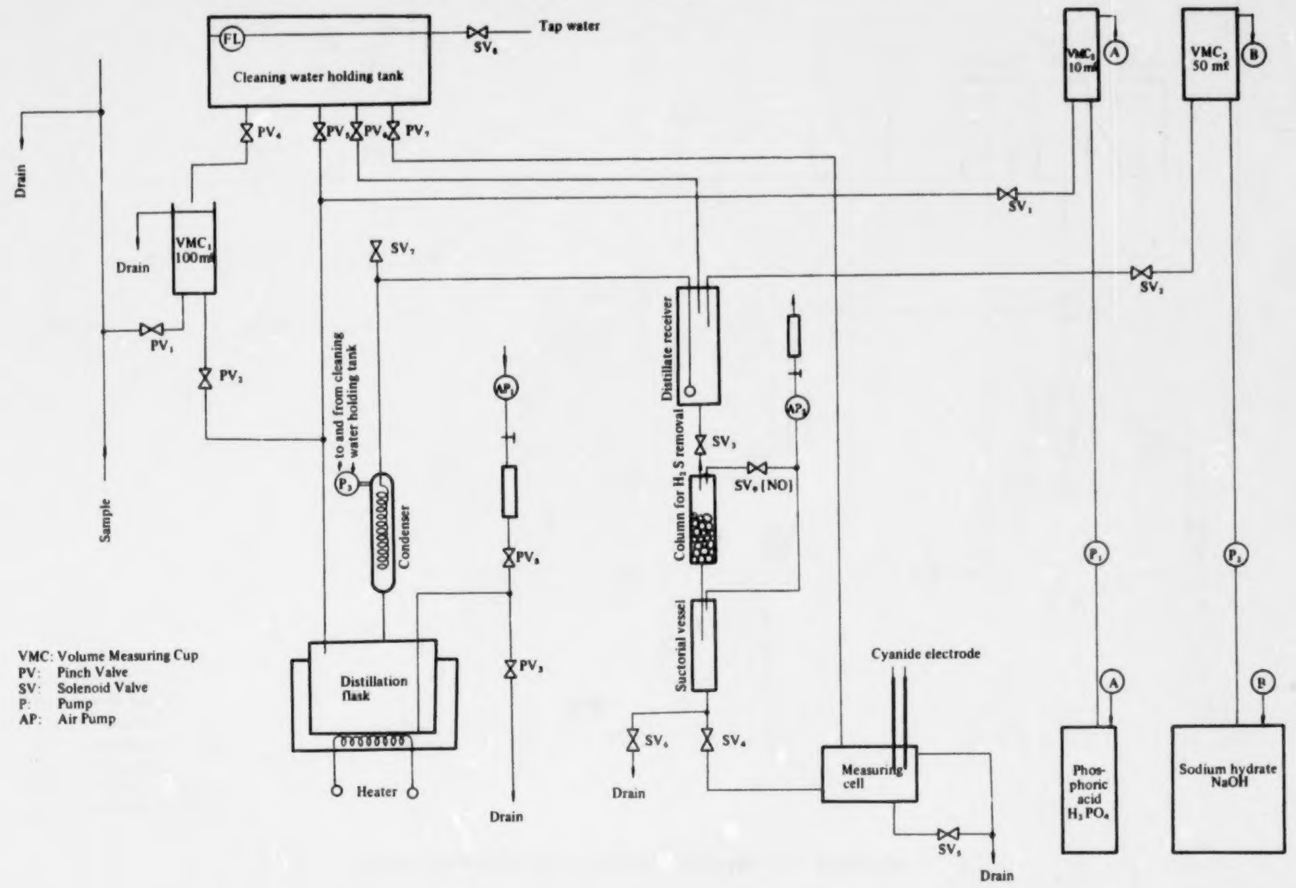
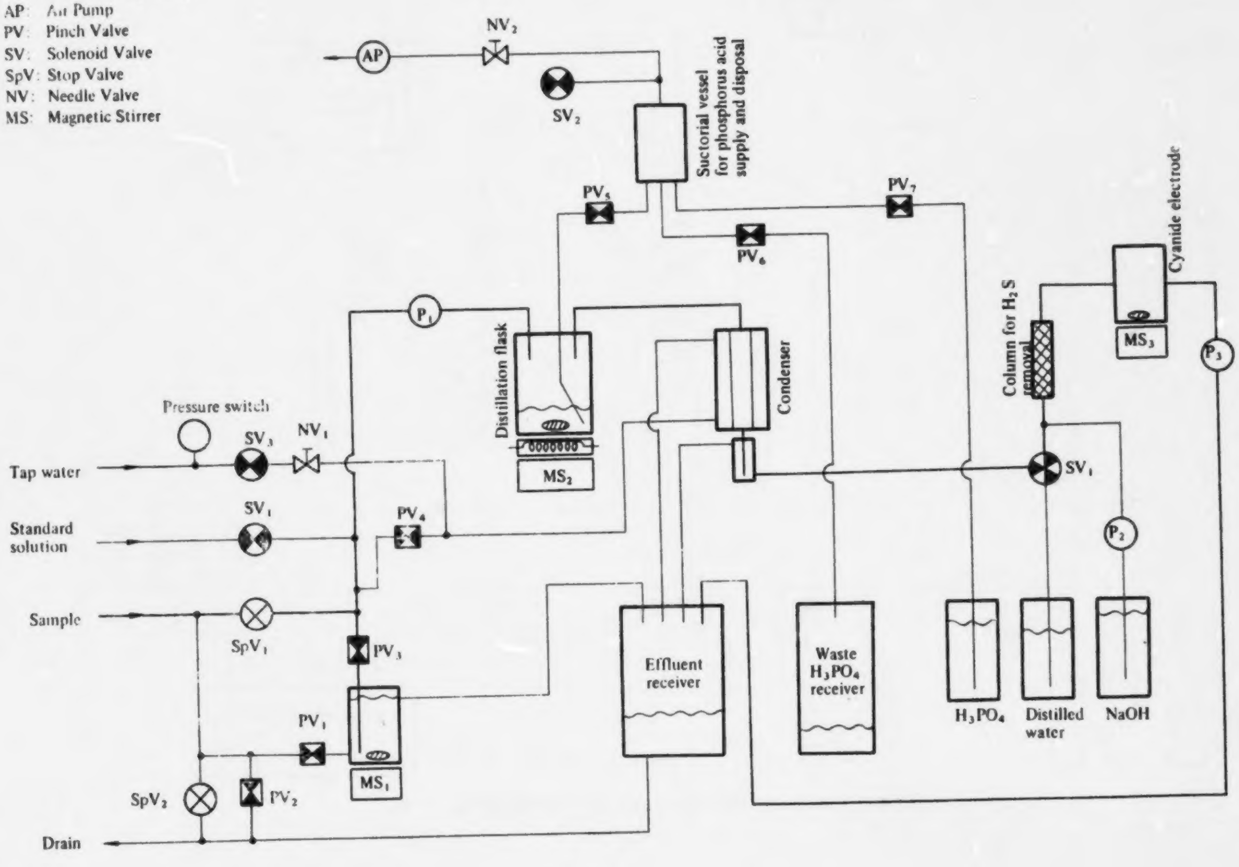


Fig. 4 Flow Diagram of Total Cyanide Monitor, Model III

P: Proportioning Pump
AP: Air Pump
PV: Pinch Valve
SV: Solenoid Valve
SpV: Stop Valve
NV: Needle Valve
MS: Magnetic Stirrer



Model II is also of a batch type. As shown in the flow diagram in Fig. 3, the system consists of distillation under acidic condition with phosphoric acid and removal of hydrogen sulfide after distillation.

100 mℓ of sample water, added with 10 mℓ of phosphoric acid, is distilled and distilled cyanide is absorbed in 2% sodium hydroxide solution in the receiving flask. The hydrogen sulfide, distilled together with hydrogen cyanide, is removed by the column either filled with the bismuth treated chilate resin or granular lead peroxide. The bismuth treated chilate resin is a chilate resin reacted with 0.4 ~ 0.5 mM bismuth per 1 (one) mℓ resin. Theoretically it has a capacity to remove equivalent hydrogen sulfide.

The use of such resin prevents the heavy metal for sulfides removal from being discharged into the effluent.

Lead peroxide oxidizes hydrogen sulfide into sulfate ion without producing any complex ion of cyanide. The time interval of measurements by this model is one hour.

Model III is of a continuous measuring type as shown in the flow diagram in Fig. 4.

200 mℓ/hr of sample water is continuously supplied into the distillation flask and distilled almost instantaneously by heated phosphoric acid in the flask. 2% sodium hydrate solution at a flow rate of 10 mℓ/hr is added to the distillate, and the cyanide is measured by an electrode. Since non-distillated substances such as minerals accumulate in the distillation flask, phosphoric acid is replaced once every day to wash the flask.

Sulfides, as is in Model II, are removed after distillation by permeating the distillate through the desulfurizing column.

These three type units, completed in December 1976, are currently undergoing field tests at an actual sewage treatment plant. The influent to the plant is mainly composed of metal-finishing wastewater and contains very small amount of domestic sewage. The water sample presently used for the field test is the chemical clarified effluent from the plant.

Major troubles found during the field test were:

(a) Distillation efficiency during night time deteriorated due to insufficient distillation heater capacity of Models I and II.

(b) Cyanide recovery rate was inconsistent owing to dew drops developing in the tube linking the condenser with the distillate receiver. and

(c) Some substance in the tap water which was used for wash water interfered the measurement. This substance is considered to be either free chlorine or the substance which forms a complex cyanide compound.

Modifications of the instruments were made, and field tests are presently conducted of the modified units.

After completing the test using clarified effluent, field tests will be continued using raw sewage as the sample.

2.2 SURFACE OIL DETECTOR

Contract research with the Association of Electrical Engineering also covers evaluation and development of surface oil detectors. Various types of oil detectors are already on the market. Two types of surface oil detectors were selected to carry out field tests for examining their applicability. One model is based on the fluorescence emitted from oils being irradiated by UV rays. The other model is based on the difference in reflectivities between oils and water. Both models have essentially the same structures. A pair of turbular floats support a light source and a light receiver, keeping a prefixed distance between the detector unit and the water surface.

In former type oil detector, an ultra-violet ray of 253.7 nm in wave length is projected and the fluorescence light is detected by a photo multiplier. It can detect crude oil, heavy oil and lubricants. However, it is not good for oils that do not produce fluorescence, such as gasoline, kerosene, vegetable oil and animal fat. As far as the detectable oils are concerned, the detector can measure the oil film thickness to a certain degree when the type of the oil is given. This detector has an automatic calibration mechanism that can correct the effects of the deterioration of light source and temperature changes.

The field test lasted for about a month and a half without producing any mechanical malfunctions thus satisfactory measurement results were obtained. Possible interferences may be detergent fluorescent and phenol contained in the sewage. However, the baseline drift during the field test was extremely small, and the interferences can be considered negligible.

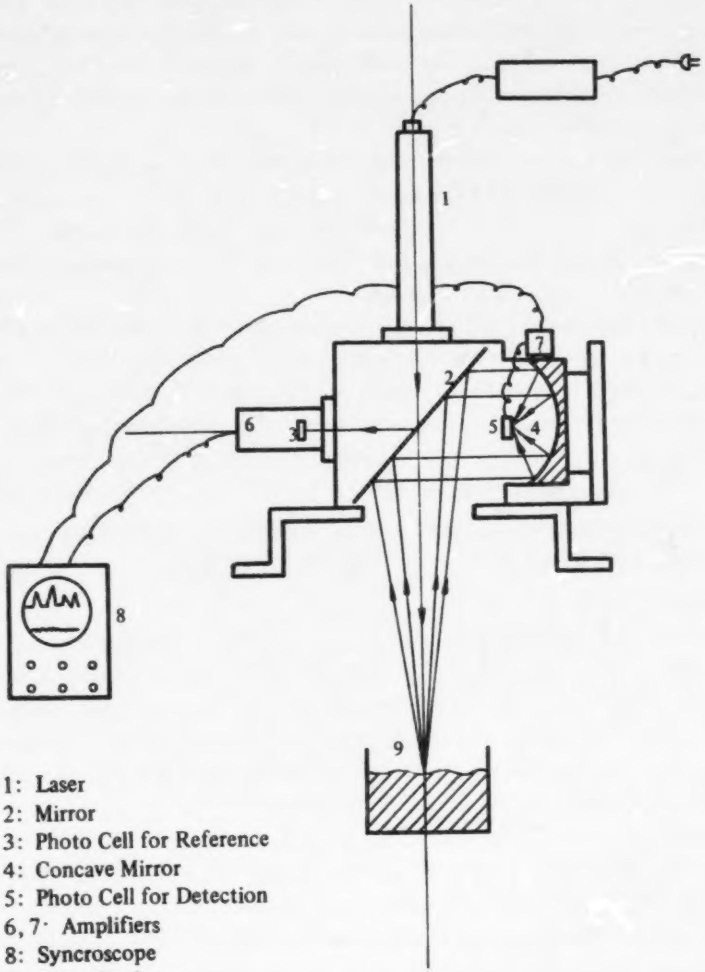
The principle of measurement of the other model is the difference in the reflectivities between water and oil. The water reflectivity is normally 2%, and that of oil is between 3.5 and 6% depending upon the type of oil. Hence, the existence of surface oil can be detected by merely measuring the reflectivity. The pulsed infrared beam is used to minimize the effect of the ambient light. Dissolved and suspended elements in the water do not interefere with the measurements.

During a month and a half long field test, no maintenance was necessary except when scum covered the whole water surface, and the performance was satisfactory.

Both oil detectors mentioned above has comparatively large floats and require a wide space to install. Therefore, it was planned to develope an oil detector which needs smaller space. The new model being developed is also based on the difference in reflectivities between water and oil but uses a laser as the light source. Since the intensity of laser beam is practically indipendent on the light path, the reflectivity measurement is scarcely affected by water level changes, which eliminates the necessity of the floats. The block diagram of the barrack model is shown in Fig. 5.

The light source is 1 mW He-Ne laser with wave length of 633 nm. The light beam from the lazer is projected on the water surface via the mirror through a transparent part at the center. A part of the light reflected on the reverse side of the mirror goes into the photo cell on the left side as a reference. The reflected light from the water surface travels to the concave mirror via the central mirror and gathers at the photo cell for detection.

Fig. 5 Block Diagram of Surface Oil Detector Using Laser Beam



1: Laser
2: Mirror
3: Photo Cell for Reference
4: Concave Mirror
5: Photo Cell for Detection
6, 7· Amplifiers
8: Syncroscope
9: Water Surface

The barrack model employs a synchroscope to read the output from the photo cells. The intensity of the laser beam is comparatively strong, so that the effect of the ambient light seems to be small. But it may be necessary to use a pulsed beam by means of a chopper to put the model into practical use.

Further studies are being done on the output system and so on. Laboratory and field tests are currently under way. This model, upon completion for practical use, will enable measurement even in narrow manholes.

2.3 FUTURE DEVELOPMENT PROJECTS

The contract research entrusted to the Association of Electrical Engineering will include studies of (a) developing total-chromium and copper monitors based on automated colorimetric analysis, (b) developing a cadmium monitor based on ion selective electrode method and (c) conducting field test to examine the feasibilities of these devices for future practical use.

With regards to total chromium and copper monitors, automated colorimetric analysis would be a practical method since regents appropriate for automation are available. Products based on this method are already on the market. Presently, researches are conducted on the pretreatment methods to maximize the dissolved fractions of chromium and copper in sewage.

Automated colorimetric analysis of cadmium does not seem to be practicable, as suitable reagents are not available yet. Ion selective electrode for cadmium, although promissing, is interfered greately by organic substances. Therefore, methods to effectively remove organic interferences are now being studied.

Furthermore, basic studies will be initiated to apply flameless atomic absorption spectometry and anode stripping voltimetry methods to water quality monitors.

3. AUTOMATIC WATER QUALITY MEASUREMENT DEVICES TO CONTROL WASTEWATER TREATMENT PROCESS

3.1 TOC ANALYZER

Nishiyama Sewage Treatment Plant is utilized as a demonstration plant to evaleate the effect of chemical coagulat addition to the aeration tank. A continuous TOC analyzer was installed at the plant on March 1976 and has been operated successfully. This TOC analyzer is an improved version based on the experimences at the Morigasaki Sewage Treatment Plant and the Arakawa River Left Bank Sewage Treatment Plant, as presented at the 4th Conference.

The major purpose of this TOC analyzer installation is to study the conversion of dissolved organics into activated sludge in the biological treatment process. However, the sampling system is designed to feed the analyzer the following four kinds of samples at 90 min. intervals in turns by an automatic sample exchanger: primary effluent, secondary effluents from both control and chemical addition systems, and filtered secondary effluent.

Suspended solids in primary effluent is removed by a compact continuous centrifuge having a bowl capacity of 3 liters. This centrifuge requires manual intermittent operation to discharge centrifuged solids every two weeks. By conducting centrifugal operation under the feed rate of 2.5 ℓ/min. at 4,000 rpm, 30 ~ 80 mg/ℓ suspended solids in the primary effluent can be lowered to 5 ~ 10 mg/ℓ.

Several minor troubles developed since commencing continuous operation of the analyzer. But none was so serious to stop the measurement, and data were obtained almost 100% of the period.

Troubles experienced so far are as follows:

(1) The tubing at the inlet and outlet of the infrared analyzer deteriorated in quality faster than expected. The replacement of the tubing can be done only by the manufacturer.

(2) The cleaning of the scrubber for IC removal is extremely difficult because of its ill-designed setup.

(3) The sample feed pump clogged often.

(4) The pipe at the outlet of the furnace corroded considerably, and should be replaced within one year.

(5) The corrosion of the inside of the furnace was observed.

(6) The humidity of the air used as the carrier gas was very high, resulting in frequent replacement of the dehumidifying silica gel.

(7) The performance of dust filter in front of the infrared analyzer was poor. Thus it was replaced by a glass wool filter which caused a new trouble of condensation of moisture.

Samples were taken from the scrubber of the continuous TOC analyzer, and TOC was measured by a laboratory-type TOC analyzer. Fig. 6 shows the comparison of TOC thus obtained with that by the continuous analyzer. Fairly good correlation is observed.

Fig. 7 shows the comparison of TOC obtained by manual sampling and analysis of primary and secondary effluents with that obtained by the continuous TOC at the same time. The black circles and triangles in the Figure indicate total TOC manually analyzed, and the white ones represent dissolved TOC manually analyzed.

According to Fig. 7, the continuous TOC analyzer gives data close to the measurement of the dissolved TOC. As to the primary effluent, suspended solids removal is done by a centrifuge as mentioned before. However, with regards to other samples, no provision is made for SS removal except a coarse filter for removal of larger materials. Therefore, a considerable portion of SS seems to be removed in the sampling and IC removal system.

Fig. 7 also indicates that the correlation between dissolved TOC by manual analyses and TOC obtained from the continuous TOC analyzer is worse than that shown in Fig. 6, implicating not only loss of SS in the sampling system but also a possible change of water quality in other form may exist.

The TOC analyzer of this type is considered most appropriate for water sample containing high SS among the continuous TOC analyzers currently on the market. However, at this stage it can be concluded that the continuous measurement of total TOC is not practicable.

Fig. 6 Comparison of TOC, Manual and Automatic Analyses
– Samples taken at the outlet of the scrubber in the TOC monitor –

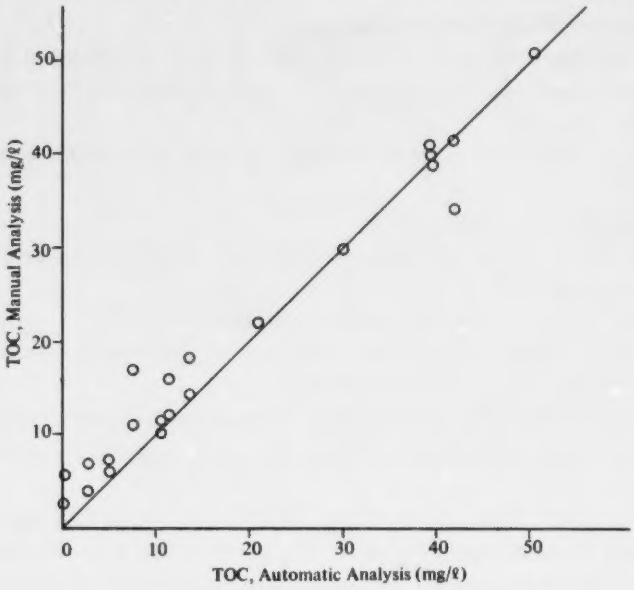
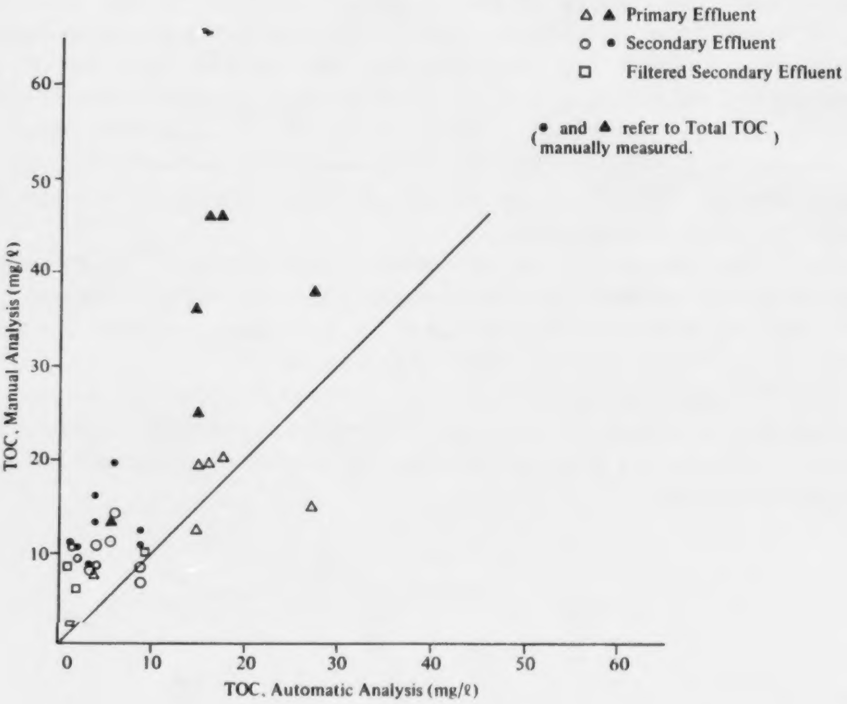


Fig. 7 Comparison of TOC, Manual and Automatic Analyses



3.2 CONTINUOUS UV PHOTOMETER

Most of the organic substances bear the nature of UV absorbance. Being based on this principle, continuous UV photometer is sold on the market as a organic content monitor.

The correlation between UV absorbance and TOC of various kinds of sewage was investigated extensively by laboratory analyses. It was found that comparatively good correlation was seen when water sample was of the same type. Therefore, continuous UV photometers were installed at the Nishiyama Demonstration Plant and Toba Pilot Plant for the field evaluation.

This continuous UV photometer measures the extinction of 254 nm UV beam. The turbidity influence is corrected by taking the ratio of transmitted light intensities of ultraviolet and visible rays. Soon after continuous operation was started, it was discovered that the ambient temperature variation affected the measurement significantly. The temperature characteristic of the detector unit is described in Fig. 8. Accordingly, the photometer was remodelled to put the detector unit in a constant temperature box of 40°C. An activated carbon column was installed at the exhaust pipe of the constant temperature box to remove ozone generated in the box.

Fig. 8 Effect of Ambient Temperature on UV Absorbance Measurement

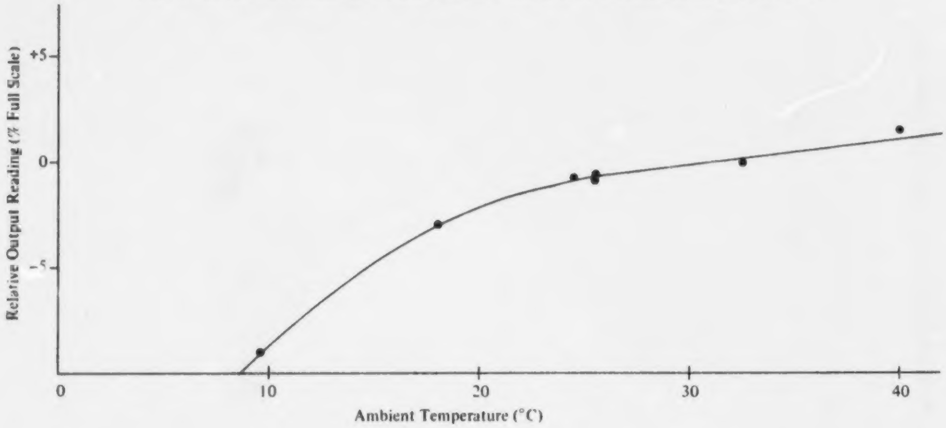


Fig. 9 shows the relationship between UV absorbance and TOC (value obtained by continuous TOC analyzer) at the Nishiyama Demonstration Plant, which seems relevant with the exponential function.

Fig. 10 shows the relationship between UV absorbance and turbidity with regards to primary effluent. Judging from this figure, there is no correlation between them, therefore, the influence of turbidity seems to be compensated.

Fig. 11 is the data obtained at the Toba Pilot Plant. Each datum represents an average of 12 values obtained from analyses of 24 hours composite samples. According to Fig. 11, the data tendencies of activated carbon effluent and other effluents differ slightly. This may be because the residual organic compounds in the activated carbon effluent are composed mainly of those with no UV absorbance, such as polisucroses.

Fig. 9 Relationship between UV Absorbance and TOC (Automatic Analyzer)

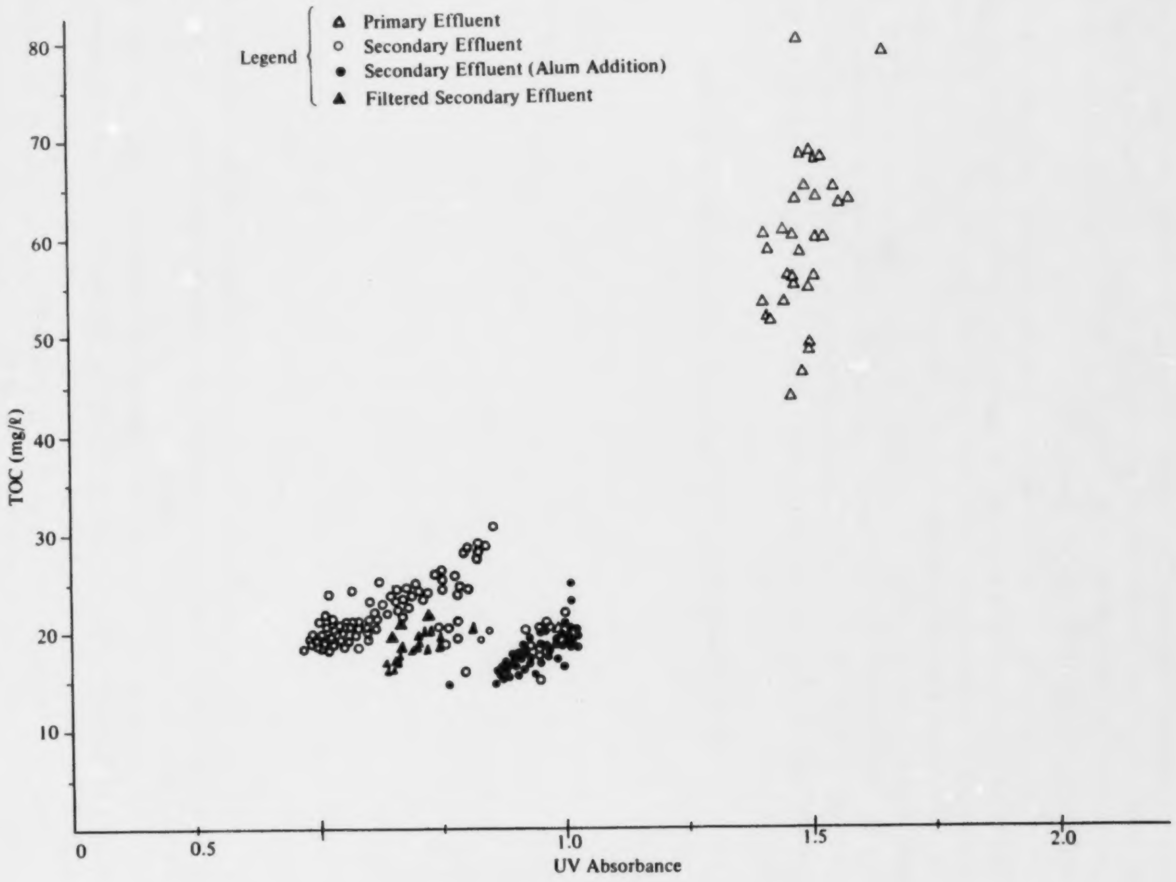


Fig. 10 Effect of Turbidity on UV Absorbance – Samples are primary effluent. –

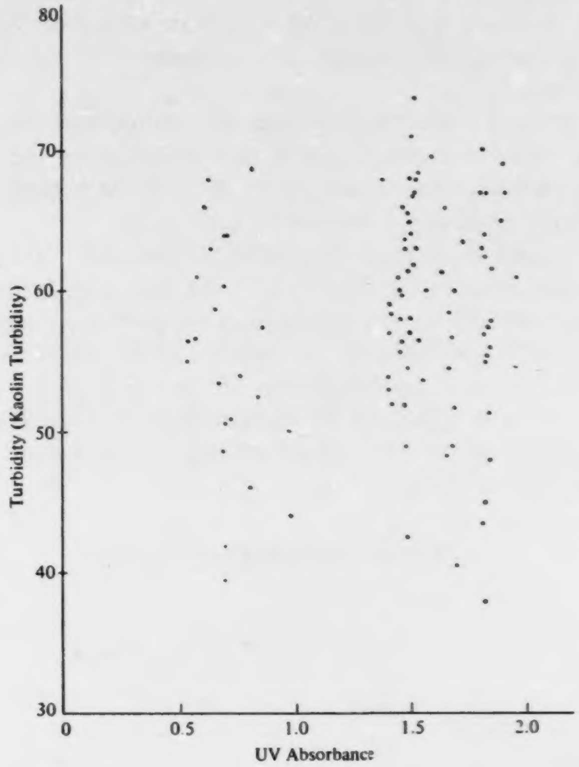
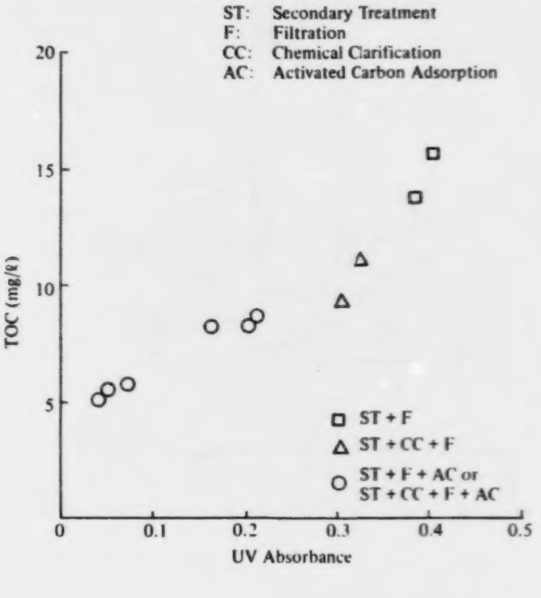


Fig. 11 Relationship between UV Absorbance and TOC, Toba Pilot Plant



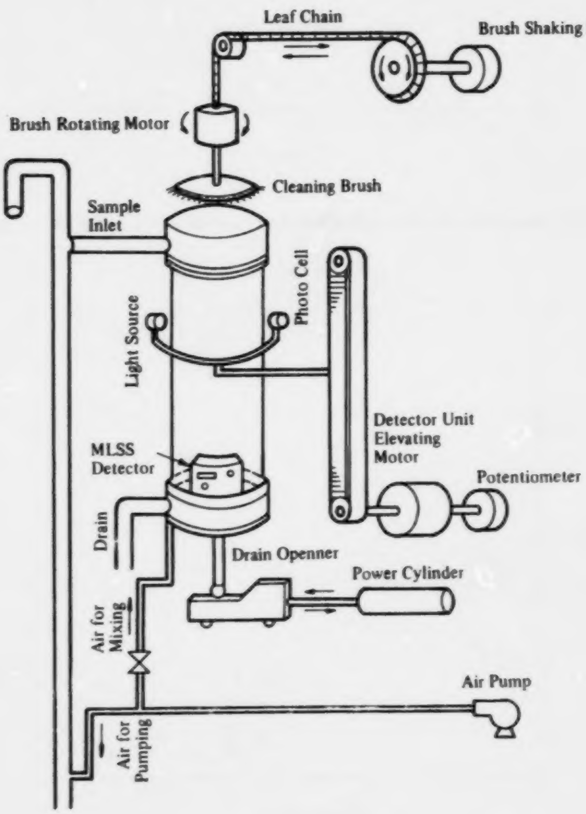
3.3 AUTOMATIC SV AND SVI METER

SV and SVI are useful indexes for operation and maintenance of the activated sludge process. A couple of SV and SVI meters are already on the market. Fig. 12 shows an example of a block diagram. The mechanicms of each SV and SVI meter is virtually the same.

After air lifting the water sample from the aeration tank into the settling tube of 170 mm in inside diameter, MLSS is first measured by the SS meter of the scattering light method with air being injected from the bottom for mixing. The meter reading is reqistered in the memorizing unit.

Next the stirring is stopped, to conduct SV measurement. Light source and photo cell installed outside the settling tube are descended by a motor until the transmitted light intensity becomes lower than the fixed level, thus the light source and the photo cell follow constantly the position of the sludge level. The SV value is detected by the potentiometer, connected to the motor, 30 min. after the start of settling. Based on these MLSS and SV measurements, SVI is computed electrically. At each measuring time, the inside of the settling tube is cleaned automatically by a brush to prevent interference by stain.

Fig. 12 Block Diagram of SV and SVI Meter



The SV and SVI meters are actually in use at many sewage treatment plants and are generally working well. Figs. 13 and 14 shows the comparisons of SV and MLSS obtained by an automatic meter and manual analyses at a sewage treatment plant in Aichi Prefecture. The figures indicate that both values agree fairly well.

It was observed, however, that color of the sewage sometimes effected the MLSS or the SVI measurements. Therefore, modification of the meter by mounting another SS meter on the top-side of the settling tube is being investigated in order to correct the effect of color variation.

Fig. 13 Comparison of SV, Manual and Automatic Measurement

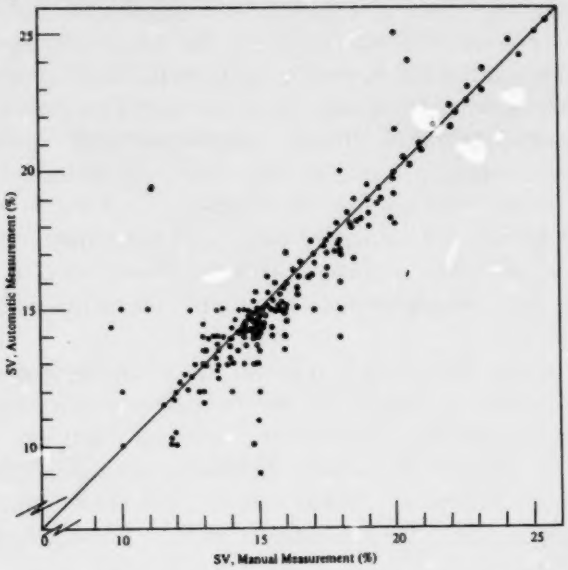
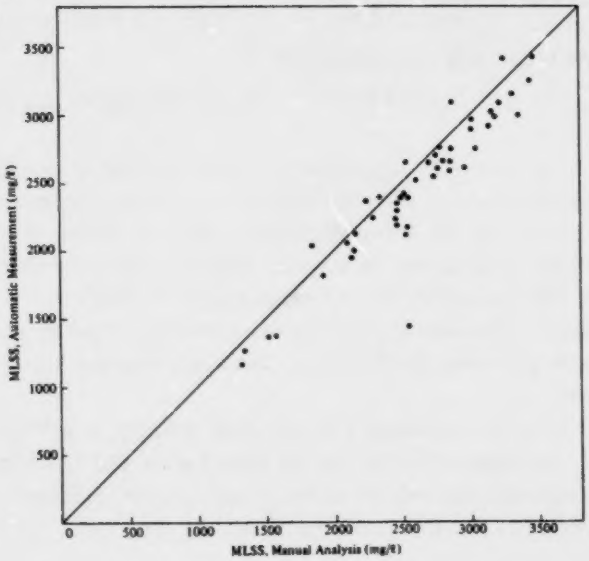


Fig. 14 Comparison of MLSS, Manual and Automatic Measurements



3.4 AUTOMATED COLORIMETRIC ANALYZER FOR PHOSPHORUS

At the Nishiyama Demonstration Plant, continuous measurement of dissolved hydrizable phosphorus in primary and secondary effluents is carried out for the purposes of mole ratio control of chemical addition and others. The measuring device basically is the same as the automated colorimetric analyzer for ammonia, which was reported at the 4th conference, except that it uses a filter paper for filtration and a hydrogen peroxide solution for cleaning.

The measuring device, installed in December, 1976, has been operated for several months. This device has an automatic calibration mechanism, and it assures certain level of accuracy. Data obtained by the automatic analyser agreed well with those attained by manual analyses. However, the device developed a number of mechanical troubles as listed below, leaving room for further improvement.

(1) For some reasons, when the flow ratio of sample and reagents or that of sample and air for separation is destroyed, unusual color development occurred.

(2) The filtration of sample water is conducted continuously by using a filter paper. The filter paper, after use, tore occasionally. Leakage of water at the filter sometimes caused entrance of suspended solids into the filtrate. The back flushing of sulfuric acid caused by back pressure opened holes in the filter paper. Various attempts are being made to correct these defects, but substantial changes are deemed necessary.

(3) In order to prevent formation of slime in the tubing, hydrogen peroxide solution was used for cleaning. But it was discovered that scales, which could not be removed by hydrogen peroxide solution, were formed inside the mixing coil. As these scales can be removed by sodium hydroxide solution, hydrogen peroxide and sodium hydrate solutions are used in turn to clean the tubing. However, alkaline solution causes unusual color development, so that it takes a long time to restart the normal operation.

(4) When replacing the pump tube, sample water, containing sulfric acid, back flowed due to internal pressure and wetted often the operators and the instrument.

3.5 OTHER MEASURING INSTRUMENTS

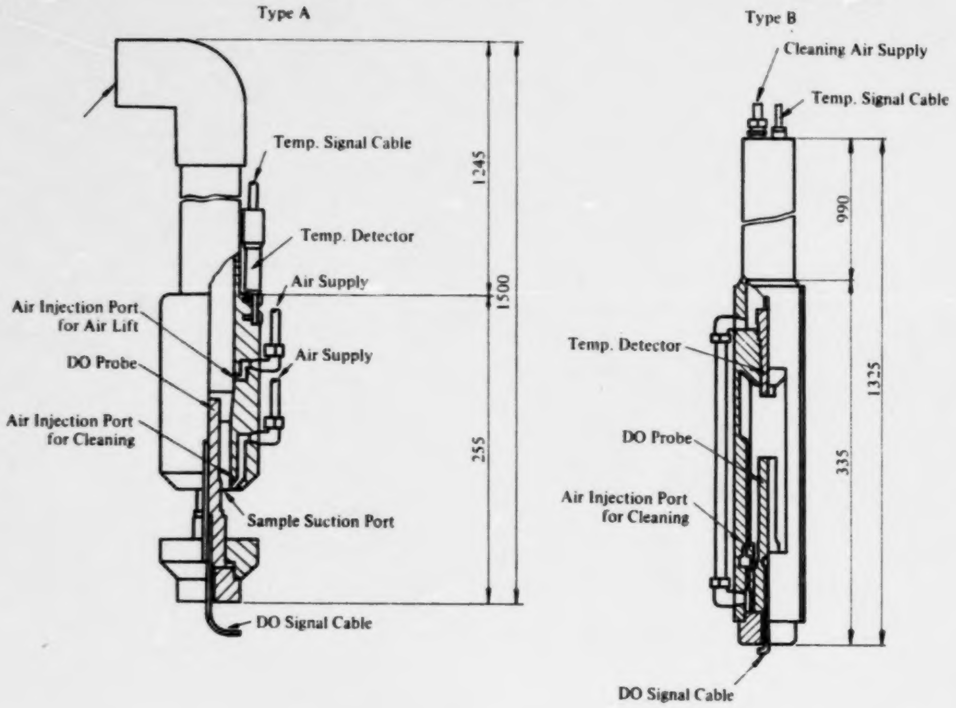
3.5.1 AUTOMATIC CLEANING UNIT FOR DISSOLVED OXYGEN ELECTRODE

When measuring dissolved oxygen in the aeration tank, it is important to prevent the formation of slime on the electrode surface. Recently, an automatic cleaning system employing air jet was introduced. Fig. 15 shows this new cleaning system. Type-A unit is designed to measure dissolved oxygen when flow velocity is insufficient for DO measurement. At measuring time, the air, from the upper air injection port, pumps up water to accelerate the velocity. And at cleaning time, the air is injected from the lower air injection port and clean the electrode surface by vigorous aggitation.

Type-B unit is for the occation that the flow velocity is sufficient for the DO measurement. It has large openings at the position of the DO electrode for the water to pass through and has only an air injection port for cleaning.

The effectiveness of this unit has been proved by field tests.

Fig. 15 DO Probe with Air Injection Cleaning System



3.5.2 ULTRASONIC WAVE SLUDGE DENSITY METER

An ultrasonic wave sludge density meter is installed at the Nishiyama Demonstration Plant. The meter, for the past two years, is continuously measuring the concentrations of primary sludge, excess sludge and return sludge. The sludge concentration agreed well with those attained by manual measurements. Especially, good results have been obtain with regard to excess sludge.

However, the variation in the sludge composition seems to pose some problems. To obtain good results in the measurement of the primary sludge, it is necessary to increase the frequency of calibration.

3.5.3 SLUDGE LEVEL METER

Sludge level meter, especially ultrasonic-meters, are becoming widely used at various sewage treatment plants for automatic sludge withdrawal control systems.

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PAPER NO. 5

STUDIES ON ADVANCED WASTE TREATMENT

APRIL 26-28, 1977
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STUDIES ON ADVANCED WASTE TREATMENT

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1. UPGRADING OF EXISTING SEWAGE TREATMENT PLANTS BY CHEMICAL ADDITION TO AERATION TANK

1.1 INTRODUCTION

In Japan there are not yet any national water quality standards established for nitrogen and phosphorus. However, eutrofication is advancing in an increasing number of lakes, bays and other waters. It is therefore presumed that a substantial number of sewage treatment plants will be required in the near future to install themselves with facilities for removing phosphorus, nitrogen. But in our country it is considered extremely difficult, in many cases, to build additional facilities in the existing sewage treatment plants due to difficulty in securing tracts of land, although the situation is different in the case of newly planned plant.

This is the reason why attention is being focused into the method to upgrade the existing plants by adding metal salts to the existing facilities such as aeration tank. This method has already been in practical use in the Great Lakes region of the United States and some parts of Europe. But the situation in Japan is somewhat different from others; the strength of sewage is weak. On the average, domestic sewage in Japan contains 150 mg/ℓ of BOD_5, 3 ~ 4 mgP/ℓ of T-P, 30 mgN/ℓ of T-N and 100 mg/ℓ of total alkalinity as $CaCO_3$.

The Public Works Research Institute, the Ministry of Construction, in collaboration with the Sewage Works Bureau of of Nagoya City, has been operating Nishiyama Sewage Treatment Plant in the city for two years for experimental purposes. Since then, experiments have been carried out by an addition of alum to the aeration tank to determine the effect of the alum addition on the entire functions of the treatment facilities. The results in the initial stage were reported at the previous, 4th conference. This report covers the performance in the ensuing period.

1.2 OUTLINE OF FACILITIES AND TESTING PROCEDURE

The Nishiyama Plant is a relatively small sewage treatment plant adopting a separate sewer system, having a design daily average flow of 20,000 m^3/day (30,000 m^3/day at maximum). Since the plant's served area is a residential area, incoming loadings fluctuate widely at times. There is no recycling of digester supernatant and other wastes from the sludge treatment facility, because sludge is treated at a separate plant. The conventional activated sludge process is adopted in the secondary treatment facility. Two bays – an alum bay and a control bay are used to compare their performance. There are three final sedimentation tanks. At the first two months, two of them were used one for the each bay. But they are now being used in a rather irregular manner – two on one and one on the other, to ensure the better quality of the plant effluent. The flow chart of the plant is shown in Fig. 1.1 and the dimension of the plant in Table 1.1.

Fig. 1.1 Flow Diagram of Nishiyama S-T-P

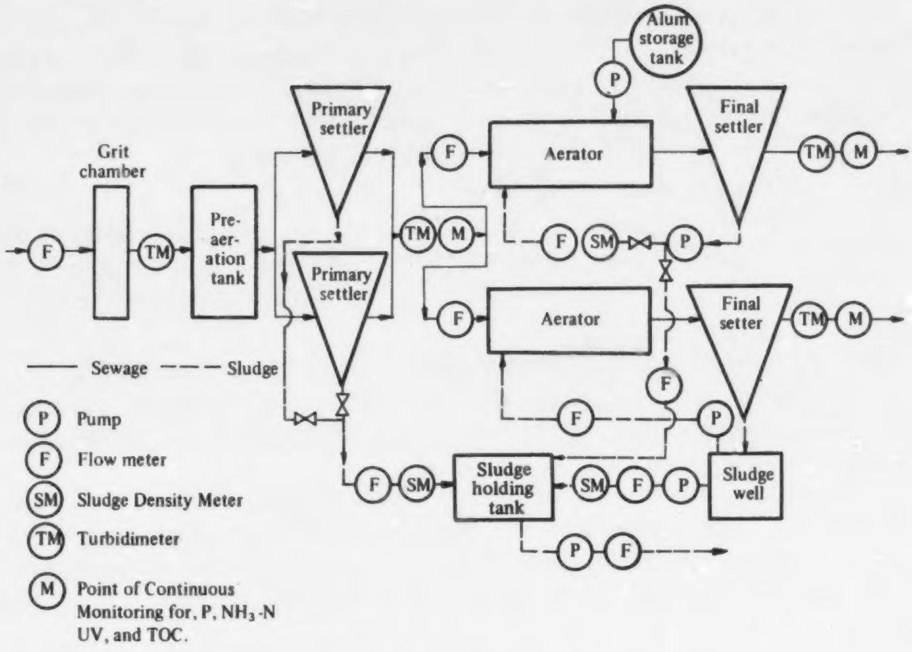


Table 1.1 Outline of Nishiyama Facilities

Item / Facility	Type	Dimension (m)	Number	Total volume (m^3)	Design detention time
Grit chamber	Rectangular	W L H 2.5 × 10 × 1.85	2		1.2 min.
Preparation tank	Diffused aeration	4.0 × 20 × 3.5	1	245	1.3 min.
Primary settler	Rectangular	5.0 × 28 × 30	4	1,680	1.3 hr.
Aerator	Diffused aeration	5.0 × 40 × 50 × 2 bay	2	4,000	3.2 hr.
Final settler	2 storied rectangular	5.0 × 1st 22.5 / 2nd 27.5 × 3.0	3	2,250	1.8 hr.

Notes: 1. Influent coming in by gravity.

2. Sludge is being treated at the adjacent plant.

1.2.1 COAGULANT AND DOSING PROCEDURE

The coagulant used is liquid alum ($Al_2(SO_4)_3 \cdot 18H_2O$) containing about 8 per cent of Al_2O_3. It is dosed at the end of the aeration tank, and the dose rate is controlled to maintain the concentration of Al at the set level by use of a proportional pump. During the operational period of about 16 months, the dose concentration was from 8 to 6, 4, 3 mg/ℓ of sewage in terms of Al (from about 100 to 75, 50, 32 mg/ℓ of sewage in terms of alum). During this period, the concentration of phosphorus in the primary effluent changed from 2 to 7 T-Pmg/ℓ in the daily average. The mole ratio of Al to P also proved to vary widely.

1.2.2 SAMPLING AND ANALYSIS

On a daily base, grab sampling is conducted every one or two hours by automatic samplers and 24 hour composite sample is adjusted in proportion to the flow rate. Samples are also taken manually as needed. Analyses are made two to four times a week.

1.2.3 AUTOMATIC CONTINUOUS WATER QUALITY ANALYZERS

The plant is equipped with several types of automatic water quality analyzers for continuous monitoring of the water quality of both influent and effluent and controls of the process.

There are ultra-sonic sludge density meters (3 units), TOC meter (1 unit, 6 points), UV meter (1 unit, 6 points), turbidimeters (5 units), phosphorus meters (3 units), ammonia nitrogen meters (3 units), PH meters (6 units) and pre-treatment units for these analyzers. Their operational performance is reported in a separate paper.

1.3 RESULTS AND DISCUSSION

The survey has been held continuously since February 1975. The period of survey, excluding the first two months, can be divided into seven experimental phases by the rate of alum dose and dosing methods. In Phase II, two of the three final sedimentation basins were used for control bay, and after Phase III, two of the three basins were used for alum bay. This switch was made because of a rise of MLSS in the alum bay which brought about massive solid wash-outs from the final settler under heavy hydraulic loading period. Summary of operational conditions in each phase is shown in Table 1.2. In Phase V, alum was dosed at a fixed rate determined by the daily average flow in order to test the situation under which flow proportional dose control is not possible. In Phase VIII, the point of addition of alum was moved from the end point to a point 3/4 of the entire length from the inlet (20 m from the terminal and about 60 min. in aeration time).

In each phase, efforts were made to maintain MLVSS of the two bays as close as possible, but this was not necessarily being achieved. Especially in the control bay, as temperatures went down, activated sludge began getting bulky, raising SS in the effluent and making it difficult to maintain MLSS of desired level. In general, MLSS was kept at low levels due to a low strength of sewage and a small capacity of the final settlers. The summary of the water qualities of influent and effuent in Phase II, IV, VI and VII is shown in Table 1.3.

Table 1.2 Summary of Operation

Phase	II		III		IV		V		VI		VII		VIII **)	
Period	1975.4.1 ~9.21		9.22~10.19		10.20~ 1976.1.7		1.8~2.7		2.7~3.31		4.1~6.30		8.23~9.2	
Item / Bay	Con-trol	Alum	Con-trol	Alum	Con-trol	Alum	Con-trol	Alum	Con-trol	*) Alum	Con-trol	Alum	Con-trol	Alum
Flow rate (m^3/d)	11,950		12,550		11,700		9,800		10,900		13,075		13,600	
Detention time (hr)	3.0	3.0	2.9	2.8	3.2	3.0	3.6	3.4	3.2	3.2	2.9	2.9	3.5	3.5
MLSS (mg/ℓ)	1,025	912	1,403	1,002	768	1,546	765	1,496	631	1,333	743	1,810	659	1,231
MLVSS (mg/ℓ)	719	589	946	634	620	1,000	673	1,136	517	968	477	1,238	432	737
MLVSS/MLSS (%)	70.1	64.6	67.4	63.3	80.7	64.7	88.0	75.9	81.8	72.6	77.5	67.0	65.5	59.8
SVI	94	112	76	89	115	72	136	76	164	68	78	58	96	61
BOD load (kg/kg/d)	0.41	0.46	0.21	0.29	0.64	0.32	0.68	0.35	0.75	0.36	0.59	0.25	0.41	0.23
SRT (d)	3.2	1.9			3.0	3.8			2.0	3.1	2.3	4.8	2.3	4.3
Settling time (hr)	3.6	1.8	1.7	3.4	1.8	3.6	2.2	4.3	2.0	3.9	1.4	2.8	1.5	3.1
Over flow rate ($m^3/d/m^2$)	23.8	47.6	50.0	25.0	46.8	23.4	39.2	19.6	43.6	21.8	52.3	26.2	54.4	27.2
Alum dose (mg/Al/ℓ)	–	8	–	8	–	6	–	4	–	4	–	3	–	6

*) Constant feed **) Point of dose was changed

Table 1.3 Summary of Performance

Item		Unit	Influent		Primary Effluent		Final Effluent (Control)		Final Effluent (Alum)		Removal Eff. Over Secondary (%)	
			Ave.	Range	Ave.	Range	Ave.	Range	Ave.	Range		
PHASE II	PH	mg/l		6.88~7.30		6.86~7.60		6.26~7.60		6.62~7.3		
	Turb.	mg/l	93.8	67.0~132.0	72.0	52.0~92.0	8.0	4.1~11.3	7.4	4.9~11.6	88.9	89.8
	SS	mg/l	110	60~114	40	36~73	8.5	6.2~12.4	15.0	4.0~57.0		
	T-Alk.	mg/l	77.3	60.0~106.5	88.7	75.5~135	36.8	12.0~95.0	31.7	13.5~42.5		
	T-BOD	mg/l	87	47~124	54	32~95	10.9	6.5~19.5	7.0	3.6~12.8	80.0	87.2
	S-BOD	mg/l	40	26~69	35	23~63	5.1	2.9~8.4	3.4	1.3~5.2	85.3	91.2
	T-COD	mg/l	59	44~90	43	27~74	14.0	9.3~16.4	9.8	8.2~18.9	67.2	77.0
	S-COD	mg/l	23	17~35	24	15~30	10.4	5.1~14.6	6.6	5.1~11.1	57.4	73.0
	T-TOC	mg/l	72	39~122	53	38~87	17.6	12.0~36	13.4	8.0~22.3	67.0	74.9
	S-TOC	mg/l	30	20~51	28	16~51	13.6	9.4~29.5	10.5	7.7~21.0	50.0	62.1
	TKN	mg/l	24.9	21.8~28.7	24.0	19.9~39.1	10.2	5.0~21.0	16.4	9.8~20.3	57.5	61.7
	NH3-N	mg/l	13.4	10.2~15.9	13.8	10.1~25.5	7.1	2.8~17.4	13.1	10.0~16.4	48.6	5.1
	NO2-N	mg/l	0.034	0.00~0.28	0.009	0.00~0.04	0.581	0.06~1.26	0.086	0.01~0.32		
	NO3-N	mg/l	0.053	0.00~0.23	0.037	0.00~0.15	4.10	0.19~8.50	0.653	0.12~1.41		
	T-P	mg/l	3.43	2.14~4.92	3.05	2.01~7.90	1.51	0.67~3.21	0.355	0.14~0.69	50.5	88.4
	Ort-P	mg/l	2.14	1.38~4.15	2.16	1.36~4.40	1.12	0.59~1.97	0.185	0.06~0.60	48.2	91.5
	S-T-P	mg/l	2.18	1.13~3.66	2.16	1.22~6.16	1.09	0.46~2.40	0.087	0.02~0.25	49.6	96.0
	T-Al	mg/l	0.81	0.56~1.20	0.50	0.30~0.80	0.26	0.25~0.30	0.86	0.39~1.30		
	E-Coli	N/ml					2700	200~9600	2000	180~8500		
	ABS		6.51	5.10~8.50	6.06	5.18~7.50	0.40	0.22~0.58	0.68	0.30~1.30	93.4	88.8
PHASE IV	PH			6.70~7.12		6.86~7.20		6.50~7.10		6.70~7.02		
	Turb.	mg/l	61.0	50.0~72.0	39.1	36.0~42.1	8.0	7.6~9.5	4.1	3.2~5.0	78.1	89.6
	SS	mg/l	128	90~174	42	34~49	12	6~20	4.7	0.2~12.3	71.1	94.3
	T-Alk.	mg/l	81.3	59~104	89.1	79.3~94.5	55.4	23.4~84.5	48	40~56		
	T-BOD	mg/l	133	97~168	79	64~102	22.7	1 12~28	4.5	2.5~6.9	71.1	94.3
	S-BOD	mg/l	47	39~57	46	38~60	4.7	2.2~6.4	2.8	1.9~4.6	89.7	95.9
	T-COD	mg/l	64	50~83	47	38~60	12.9	10.1~15.1	7.9	6.2~10.6	72.6	83.2
	S-COD	mg/l	20	16~28	22	17~31	9.1	6.8~10.8	6.1	4.7~6.9	50.5	72.2
	T-TOC	mg/l	75	42~110	52	25~71	18.4	6.2~34	10.9	4.0~17.0	64.7	79.1
	S-TOC	mg/l	23	12~32	23	12~32	9.7	4.3~15.0	6.6	3.5~9.0	57.5	71.9
	TKN	mg/l	26.0	24.2~34.6	23.7	21.0~28.2	15.0	6.9~20.2	17.1	14.2~20.4	32.8	27.9
	NH3-N	mg/l	13.7	12.0~14.6	15.0	13.2~16.5	11.3	5.4~15.3	14.5	12.3~16.5	24.7	3.4
	NO2-N	mg/l	0.023	0.00~0.05	0.01	0.00~0.02	0.243	0.12~0.30	0.035	0.02~0.08		
	NO3-N	mg/l	0.25	0.00~0.82	0.133	0.00~0.41	3.03	0.18~6.45	0.488	0.07~0.83		
	T-P	mg/l	3.15	2.06~48.8	3.00	1.86~3.86	1.93	1.29~2.73	0.168	0.14~0.21	35.4	34.4
	Ort-P	mg/l	1.92	1.57~2.10	2.04	1.74~2.39	1.17	0.18~1.58	0.093	0.05~0.19	42.7	95.5
	S-T-P	mg/l	1.71	1.41~2.0	1.71	1.53~1.98	1.32	1.05~1.47	0.045	0.01~0.09	22.9	97.4
	E-Coli	N/ml					5100	2500~7000	500	30~1800		
	ABS	mg/l	6.25	4.6~7.7	6.37	5.4~7.3	0.67	0.25~0.90	0.575	0.24~0.80	89.5	91.0
PHASE VI	PH			7.0~7.27		7.0~7.18		7.06~7.23		7.07~7.10		
	Turb.	mg/l	98.5	79.0~120.0	51.7	49.0~54.0	8.5	5.7~7.0	5.3	3.7~7.0	85.6	89.8
	SS	mg/l	214	56~471	45	27~54	16.2	6.8~32.0	13.9	6.0~28		
	T-Alk.	mg/l	92.3	82~99.5	90.6	88~93.5	87.5	82~94.5	73.8	67.5~82		
	T-BOD	mg/l	182	151~210	82	75~88	16.3	12.1~23	8.5	7.4~9.3	80.1	79.6
	S-BOD	mg/l	62	48~69	52	42~64	5.3	4.7~5.8	4.7	4.3~4.9	89.9	91.1
	T-COD	mg/l	87	72~120	57	48~76	15.3	12.9~21	12.4	10.4~19	73.1	70.2
	S-COD	mg/l	30	21~45	30	21~41	9.6	7.5~12.7	8.6	5.7~13.1	67.6	71.0
	TKN	mg/l	29.8	25.0~35.5	25.5	21.3~28.8	23.1	15.1~30	20.0	15.2~24	9.5	21.6
	NH3-N	mg/l	14.9	13.6~15.5	16.2	14.6~18.0	14.8	13.4~17	15.9	13.9~18	8.7	1.9
	NO2-N	mg/l	0.03	0.01~0.05	0.04	0.03~0.04	0.06	0.02~0.10	0.06	0.03~0.09		
	NO3-N	mg/l	0.08	0.00~0.23	0.04	0.00~0.13	0.20	0.00~0.32	0.16	0.00~0.25		
	T-P	mg/l	4.43	4.00~5.24	3.44	2.84~3.81	1.83	1.63~1.9	0.52	0.37~0.72	46.9	84.9
	Ort-P	mg/l	2.61	1.97~4.12	2.11	1.89~2.30	1.60	1.43~1.82	0.27	0.09~0.69	24.2	87.3
	S-T-P	mg/l	2.35	2.18~2.58	2.35	2.04~2.86	1.51	1.35~1.61	0.19	0.03~0.29	35.8	92.0
	ABS	mg/l	6.46	5.45~7.40	5.83	5.20~6.40	0.59	0.52~0.66	1.34	0.84~1.76	89.9	77.1
PHASE VII	PH			6.87~7.0		6.90~7.0		6.65~6.96		6.73~7.04		
	Turb.	mg/l			48.7	28~100	10.5	3.4~50.3	6.8	2.2~28.1	78.5	86.1
	SS	mg/l	141	96~192	45	22~84	13	4~21	11	6~13	71.1	75.6
	T-ALK	mg/l	66.8	55~80	77	68~87	46	30~69	61	50~67		
	T-BOD	mg/l	108	88~149	68	62~96	26	14~34	12	6~16	61.8	82.4
	S-BOD	mg/l	40	23~53	35	23~48	7.2	6.3~8.8	6.1	5~8	79.5	82.6
	T-COD	mg/l	61	47~76	38	29~53	13.8	11.2~16.7	12.3	10.5~14.1	63.7	64.9
	S-COD	mg/l	17	14~20	18	13~35	8.6	6.8~10.5	7.2	6.3~8.6	52.3	60.0
	T-TOC	mg/l	63	53~69	41	26~48	13.9	11~17	10.8	9.0~12.0	66.1	73.7
	S-TOC	mg/l	20	13~24	19	13~22	9.5	8.3~10.9				
	TKN	mg/l	22.8	14.6~32.1	20.0	14.5~23.0	11.9	8.6~17.9	15.2	11.0~20.3	41.5	24.0
	NH3-N	mg/l	10.2	6.0~12.9	11.7	8.0~14.8	8.0	5.2~13.7	10.7	7.6~14.5	31.7	9.6
	NO2-N	mg/l	0.07	0.00~0.22	0.05	0.00~0.13	0.29	0.22~0.54	0.09	0.08~0.22		
	NO3-N	mg/l	0.13	0.07~0.32	0.10	0.05~0.15	1.99	0.10~3.89	0.71	0.11~1.27		
	T-P	mg/l	3.50	2.92~4.06	3.04	2.73~3.58	1.68	1.13~2.27	1.09	0.53~1.96	44.8	64.2
	Ort-P	mg/l	1.67	1.25~2.03	1.81	1.53~2.28	1.11	0.36~1.74	0.61	0.17~1.45	38.7	66.3
	S-T-P	mg/l	1.79	1.10~2.23	1.88	1.26~2.51	1.12	0.18~1.82	0.40	0.02~1.53	51.5	79.8
	Ecoli	N/ml					1800	480~4500	840	200~1800		
	ABS	mg/l	4.8	2.5~9.5	4.4	2.5~6.0	0.54	0.26~0.99	0.58	0.3~0.78	87.8	86.9

1.3.1 REMOVAL OF PHOSPHORUS

a. Phosphorus Removal and Mole Ratio

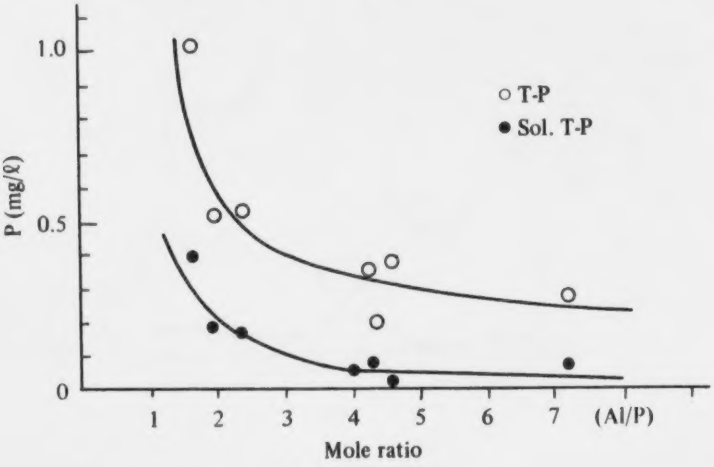
Phosphorus removal rate is raised drastically by an addition of alum. Table 1.4 shows the phosphorus removal efficiency in each phase. Fig. 1.2 shows relationships between the mole ratio of dosed Al to soluble phosphorus and residual phosphorus in effluent.

Table 1.4 Summary of Phosphorus Removal

Phase Item	II	III	IV	V	VI	VII	VIII
Primary Eff. T-P (mg/ℓ)	3.05	2.15	3.00	3.04	3.44	3.04	2.29
Primary Sol. T-P (mg/ℓ)	2.16	1.28	1.71	1.95	2.35	1.99	1.52
Aℓ Dose (mg/ℓ)	8	8	6	4	4	3	6
Aℓ/P Mole Ratio (T-P Base)	2.98	4.25	2.29	1.49	1.38	1.16	3.01
Aℓ/P Mole Ratio (ST-P Base)	4.25	7.18	4.03	2.35	1.95	1.74	4.53
Effluent T-P (mg/ℓ)	0.355	0.27	0.168	0.53	0.52	1.09	0.42
Effluent Sol. T-P (mg/ℓ)	0.087	0.09	0.045	0.17	0.19	0.40	0.03

Note: All values are average through the each phase.

Fig. 1.2 Residual Phosphorus V.S. Al/P Mole Ratio (S-T-P Base)



The residual phosphorus is 0.5 mgP/ℓ in T-P and below 0.2 mgP/ℓ in soluble T-P in phases other than Phase VII in which the mole ratio was low. Fig. 1.2 shows that if the mole ratio is above 2 (Al 4 or alum about 50 mg/ℓ), effluent soluble phosphorus can be decreased below 0.2 mg/ℓ.

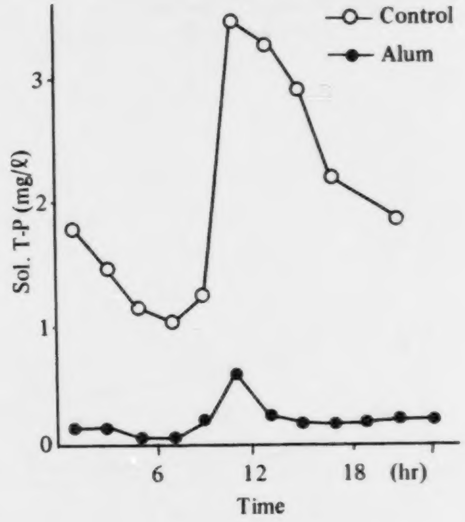
The residual phosphorus is 0.5 mg P/1 in T-P and below 0.2mg P/1 in soluble T-P in phases other than Phase VII in which the mole ratio was low. Figure 1-2 shows that if the mole ratio is above 2 (Al 4 or alum about 50mg/1), effluent soluble phosphorus can be decreased below 0.2mg/1.

Effluent PH varies with the rate of alum addition in the range from 6.6 to 7. It is not wise to dose alum at a mole ratio of more than 4, since the solubility of $AlPO_4$ is 0.01 mgP/ℓ at PH6 and 0.3 mgP/ℓ at PH7. However, as for total phosphorus removal a higher mole ratio seems to be effective in regards to floc formation. In the treatment of domestic sewage in our country where the daily average of phosphorus concentration is 3 ~ 4 mgP/ℓ, an alum dose rate of 50 ~ 70 mg/ℓ would be enough to lower T-P below 0.5 mg/ℓ and soluble phosphorus 0.2 mg/ℓ. Incidentally, at this particular sewage treatment plant, the phosphorus removal rate in the control bay is rather high with 40% ~ 50% probably due to a luxury uptake of bio-mass.

b. Phosphorus Removal Capability of Return Sludge

By an alum addition, the return sludge proves to remove (absorb) a substantial amount of phosphorus from the influent. Fig. 1.3 shows a comparison of hourly changes in the phosphorus concentration of superatant of the mixed liquer immediately after return sludge was mixed with the influent in both control and alum bays.

Fig. 1.3 Diurnal Change in S.T.P. of Mixed Liquor Supernatant



(Phase IV) In this instance, a contact with return sludge removes about 80% of phosphorus. This is considered to be absorption of aluminum hydroxide contained in return sludge. Fig. 1.4 shows the results of a jar test in which the mixing ratio of return sludge and influent was changed in order to ascertain phosphorus removal capability. As the mixing ratio is raised, as shown in the figure, the rate of removing soluble phosphorus from the superatant will increase, and the removal rate reaches 90% when MLSS is above 3,000 mg/ℓ (Al concentration of sludge 10% Al/TS).

A similar test was conducted using alum sludge yielded in the water purification plant. Aluminum in sludge was 12.5% Al/TS. Fig. 1.5 shows the results of the jar test. As in the case of return sludge, a high phosphorus removal rate is attained with the alum sludge of the purification plant. This indicates an important role that alum sludge could play in removing phosphorus within the treatment process.

Fig. 1.4 P Removal Capability of Return Sludges

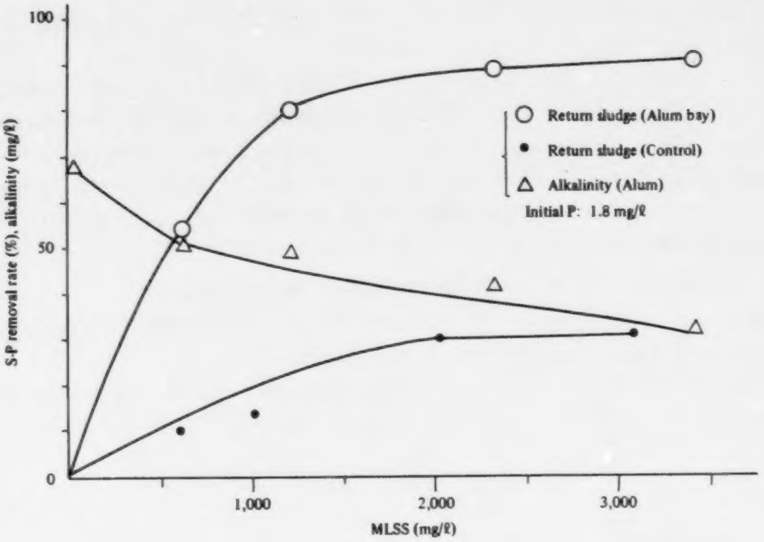
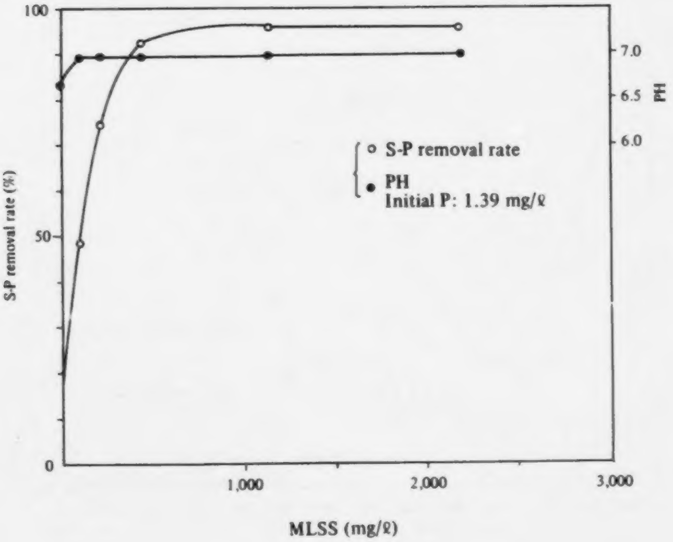


Fig. 1.5 P Removal Capability of Alum Sludge Yielded at Water Plant



Similar phenomena are observed in the pilot plants in Yokosuka and Kyoto treating secondary effluent with alum. They have stably attained a high phosphorus removal rate by returning sludge to the head of the plant.

As earlier stated, in Phase IV alum was dosed not in proportion to the inflow but a constant rate. As shown in Table 1.4, there seems little difference in phosphorus removal efficiency between flow proportional feed control and constant feed of alum. This indicates that the "secondary" phosphorus removing capability pertinent to return sludge plays an important role in overall phosphorus removal. In adding alum to the aeration tank, it would be more efficient and costly to use waste activated sludge from the alum bay for return sludge in other bays.

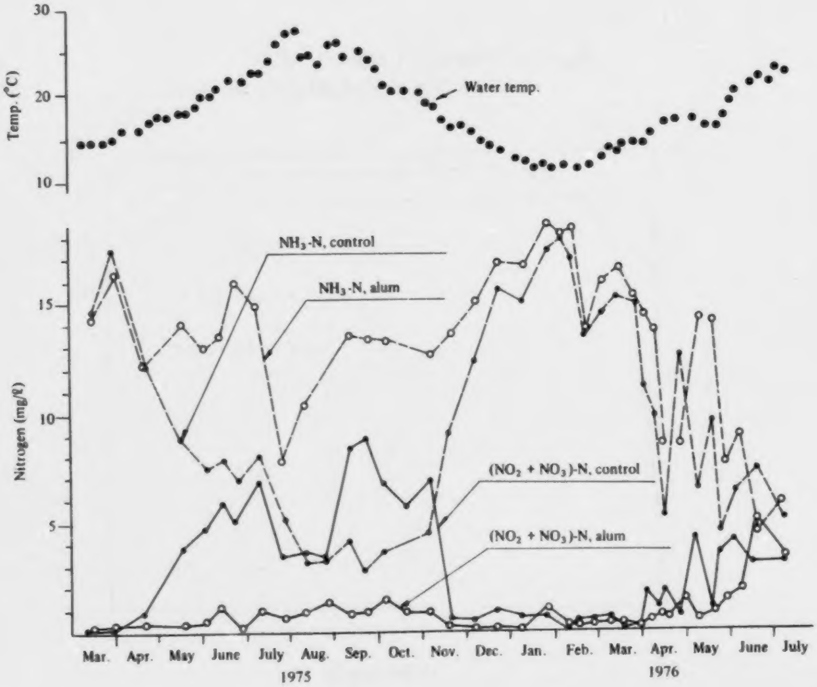
The Nishiyama plant is in the initial stage of practicing the mole ratio feed control by use of signals from automatic continuous phosphorus analyzers. If the phosphorus removal rate is determined by the mole ratio of phosphorus in the superatant of mixed liquer at the point of addition, phosphorus removal capability of return sludge could be significant and affect the mole ratio feed control itself when influent phosphorus concentration is used as a base of P.

1.3.2 NITRIFICATION AND NITROGEN REMOVAL

In this project, removal of ammonia nitrogen by nitrification is just as important an objective as well as removal of phosphorus.

Fig. 1.6 shows changes in ammonia nitrogen, nitrite and nitrate in effluent of both bays as observed during a period from March 1975 to July 1976.

Fig. 1.6 Change in Nitrogen in Final Effluent



SRT was maintained in the range of 2 ~ 5 days. In both bays nitrification did not take place in winter when temperature was low, probably because SRT was not enough. In Phases II ~ IV in which temperatures were above 20°C, substantial nitrification took place only in the control bay, while nitrite and nitrate nitrogen was slightly observed in the alum bay.

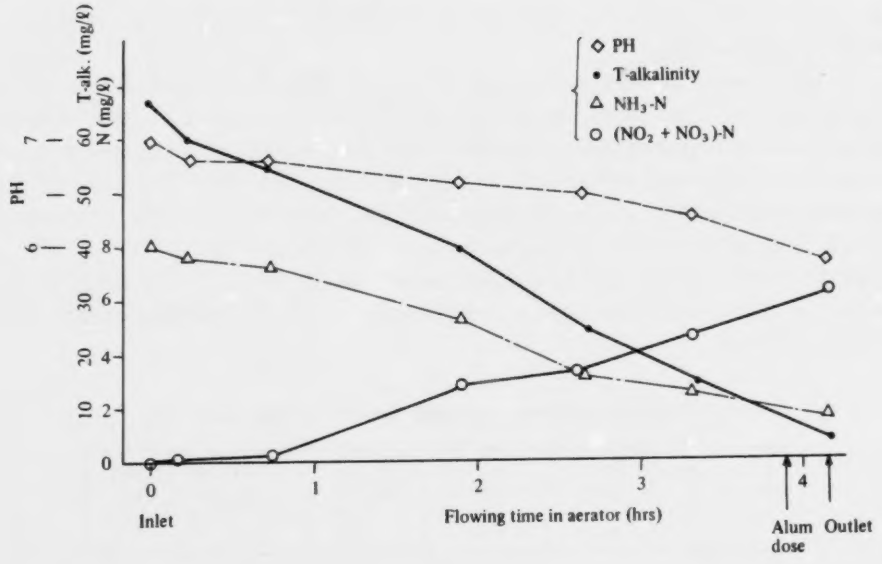
The absence of nitrification in the alum bay has been discussed from various angles; first, SRT required to maintain nitrifiers; second, interference of aluminum on the activity of nitrifiers; and third, depletion of alkalinity.

As to the interference of aluminum on nitrification, batch type laboratory tests show that an addition of 10 mgAl/ℓ (120 mg/ℓ of alum) causes no noticeable affect.

The fact that in Phase VII in which SRT is maintained higher (4.8 days), nitrification took place even in the alum bay seems to indicate that SRT has a large effect on nitrification, in conjunction with the effect of temperature. This, however, seems to have some relationships with a relatively low alum dose rate, which means a low consumption of alkalinity by alum addition. The nitrification of N1 consumes alkalinity by 7.1 (as $CaCO_3$). The influent of this plant shows an daily average T-N value of more than 25 mgN/ℓ and the alkalinity is a high 80 mg/ℓ. Complete nitrification, therefore, could not be expected without an addition of alkalis, even if other conditions are satisfactory.

Fig. 1.7 shows how the nitrogen, the PH and the alkalinity of the superatant of the mixed liquid will change as it flows down through the tank in the alum bay (Phase VII). In this instance of low loading night hours, the concentration of ammonia nitrogen is low. But complete nitrification does not take place as alkalinity is consumed by nitrification and alum addition.

Fig. 1.7 Change in Supernatant Quality in the Aerator (Alum Bay)



Thus, as in the case of non-chemical addition, the lowering in water temperature causes a problem in nitrifying the effluent in the alum bay, as shown in Fig. 1.6. Therefore, to keep desirable SRT is a major factor in conjunction with the capacity and structural improvement of the final sedimentation basin. It is also clear that complete nitrification cannot be achieved without adding alkalis in place where alkalinity is low, like the domestic sewage treatment of our country.

In the case where nitrified effluent is obtainable, it is known that fine scum appears to the surface of the final settler as a result of the generation of nitrogen gas by denitrification at the bottom of the settler. Through the experiment, it was observed during the nitrificastin in the control bay. An experiment in resettling by spraying of the effluent has been conducted. Since scum often flows out with the effluent, the installation of a scum collector is under consideration.

On the other hand, in Phase VII in which nitrification took place in the alum bay, the scum as mentioned above does not float at all and the surface of the settler is clear in the final settler, even if the level of oxidized nitrogen in the effluent is the same as that in the control level. There seem to be factors hampering denitrification in the alum dosing process.

In ordinary cases, at the plants where ntrification is taking place, there appear differences between the inflow and outflow of T-N, which results in a relatively high T-N removal rate probably due to denitrification. The T-N removal rate is generally lower in the alum bay than in the control bay even when same degree of nitrification is taking place. This is considered to have something to do with preventing the floating of scum.

1.3.3 ACCELERATION OF NITRIFICATION BY PH ADJUSTMENT

As mentioned earlier, when alum is added, it consumes alkalinity and prevents complete nitrification. Therefore, alkalis, such as sodium hydroxide and lime, need to be added in order to remove phosphorus and nitrify the effluent simultaneously in the system. There, it is difficult to use lime because of scale formation in pipes and other places. Therefore, sodium hydroxide will probably be better to be adopted for the purpose.

A bench scale laboratory test was conducted prior to the addition of sodium hydroxide to the main plant. About two months of operation demonstrated that it is possible to obtain an almost completely nitrified effluent by adding alum at the end section of the tank and sodium hydroxide at the inlet. Table 1.5, 1.6 show the operating conditions of the bench scale plant and the qualities of the influent and effluent. Almost complete nitrification and a T-N removal rate of about 70% are achieved by maintaining SRT at 8.2 days and PH at the inlet at 8.2. The problem of this process seems to be a somewhat low phosphorus removal rate due to rises in the PH of the effluent.

Table 1.5 Summary of Bench Scale Plant Operation

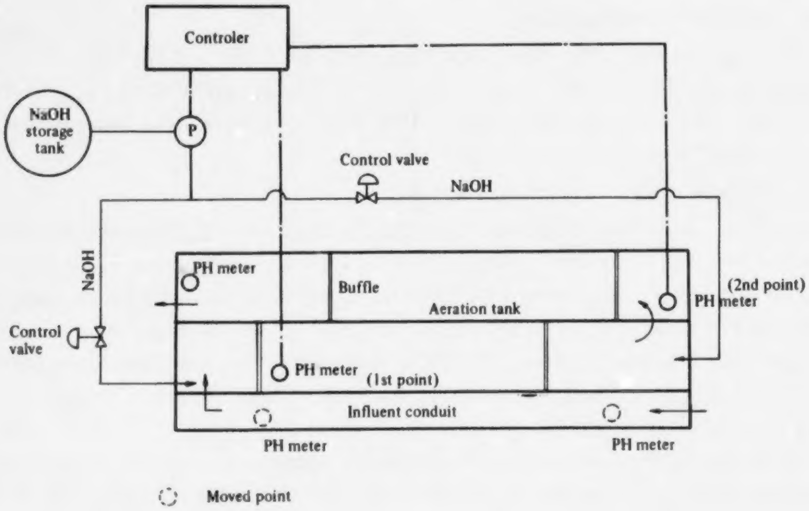
Flow (m^3/d)	Al Dose (mg Al/ℓ)	Adjusted PH	MLSS (mg/ℓ)	BOD Load (kg/kg/d)	SRT (d)
2.5	≒4	8.2	3.030	0.17	8.2

Table 1.6 Summary of Bench Scale Plant Performance

		PH	T-alkalinity (mg/ℓ $CaCO_3$)	SS (mg/ℓ)	COD (mg/ℓ)	TKN (mg/ℓ)	NH_3-N (mg/ℓ)	NO_2+NO_3-N (mg/ℓ)	T-P (mg/ℓ)
Inf.	Average	6.6~7.2	66	38	39	16.7	8.3	0.05	2.66
	Range		57~104	13~117	25~73	12~30	6~11	0~0.9	1.56~5.15
Efl.	Average	7.3~7.6	93	14	8	1.45	0.26	2.96	0.78
	Range		88~102	9~20	6~15	1.1~1.7	0.14~0.40	2.8~4.2	0.5~1.2

The addition of sodium hydroxide to the actual facilities began in mid December 1976. Since the aeration tank is of a conventional plug flow type, the caustic addition is controled at two points. One point is immediately after the mixture of the influent with return sludge and the other at the middle section of the tank. Fig. 1.8 shows PH controls chart.

Fig. 1.8 PH Control Diagram



About six weeks have passed since the caustic addition began. The average MLSS concentration within the tank has kept 3,500 mg/ℓ (SRT approx. 8 days). The dose rate of alum is 8 mg/ℓ in terms of aluminum. The addition of sodium hydroxide (20% solution of NaOH) is controlled to raise PH at the first adjustment point up to 7.8 ~ 8.0. At the present stage, nitrification has not taken place as originally expected, because the water temperature is low - 10° ~ 12°C and acclimation to change in PH is insufficient.

Two problems have occurred as to the PH adjustment at the actual facilities. One problem is that PH rises momentarily to about 12 in limited portions around the inlet. The second problem is that as a result of this, a sticky foam substances appears to the surface of the aeration tank and that scum forms on the surface of the final settler.

The dose of soidum hydroxide is controlled using PH meters in the way that the dosing pump turns on at the set upper value (8.0) and turns off at the lower value by the signals of PH meters and that the dose rate is proportional to the inflow. There is a 15-minute time lag between the dose of sodium hydroxide and the response of the PH meter. This has been found to allow the dose of surplus sodium hydroxide and raise PH around some portion of the inlet up to 12.

A separately conducted indoor experiment shows that as PH exceeds 11, nitrogen in activated sludge resolubilizes and foam formation develops. Also at the batch test using return sludge of this treatment plant, as the PH of return sludge was raised to 12 organic substances in activated sludge resolubilizes into the superatant, such high concentration as TOC of 1,000 mg/ℓ and poly-saccharide of 600 mg/ℓ. Thus, it was considered that this might have been the reason of upset of the entire process.

As a result, the point of sodium hydroxide addition was moved to the upper end of the influent waterway (approx. 40 meters); the method was changed to adjust PH at the tank inlet; and the outflow of the dosing pump was changed to a constant rate.

1.3.4 ORGANIC REMOVAL

The addition of alum raised removal rates for BOD_5, TOC and COD. To compared with control bay, the removal rate for BOD_5 rose by 30%~80%, that for COD 20% ~ 40% and that for TOC 30% ~ 40%. Similar results were attained for the removal of soluble organic substances.

1.3.5 SUSPENDED SOLID REMOVAL

In Phase II, effluent SS was higher in the alum bay. But it outrated that of the control bay after Phase III in which two final settlers were used. The debris of aluminum hydroxide floc tends to flow out more easily, in addition to rises in solid loading in the final settler. Therefore, it is necessary to provide a lower overflow rate than the ordinary design value. It is observed that as the alum dose rate is lowered, the density of the effluent SS tends to increase; this is probably because a decline in the dose rate will aggravate coagulation.

In Phase VIII in which the point of alum addition was moved to a point 3/4 of the entire length (20 meters from the end), the effluent became a little bit milky white as time passed, although there were no noticable differences in the SS value between this and other phases. The change in the effluent colour has already been reported in other studies. As reported in some of them, it seems to be reasonable to assume that this has something to do with the point of alum addition. It is considered that a deflocculation caused by over-aeration after alum addition has a significant effect on this because such a phenomenon did not take place at all in experiments in which alum was dosed from the end point.

Table 1.7 is a comparison of the composition of condensed these substances and that of solids in effluent from the control bay.

In the control bay, substances of obviously different quality compared with activated sludge flows out, but in the alum bay, activated sludge including aluminum itself comes out in broken-up form.

Table 1.7 Component of Solids

	SS in Eff. (Control)	Return Sludge (Control)	SS in Eff. (Alum)	Return Sludge (Alum)
VSS/TS (%)	49.2	70.0	40.8	63.0
TKN/TS (%)	3.79	7.28	6.75	5.85
T-P/TS (%)	0.61	2.95	2.96	3.14
Aℓ/TS (%)	0	1.74	7.65	8.10

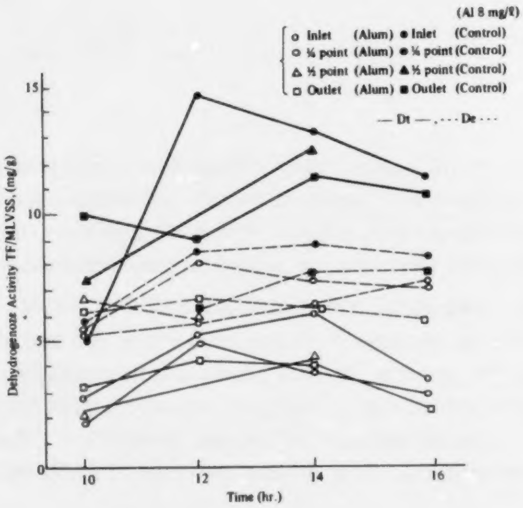
1.3.6 EFFECTS ON MICROORGANISMS

The effects alum have on activated sludge protozoa have been discussed from many angles.

In this project, survey was conducted from two points. One is the microscoptic examination of activated sludge protozoa. The other is the measurement of dehydrogenizing activity, which is considered to have relationships with the metabolic activity of micro-organisms.

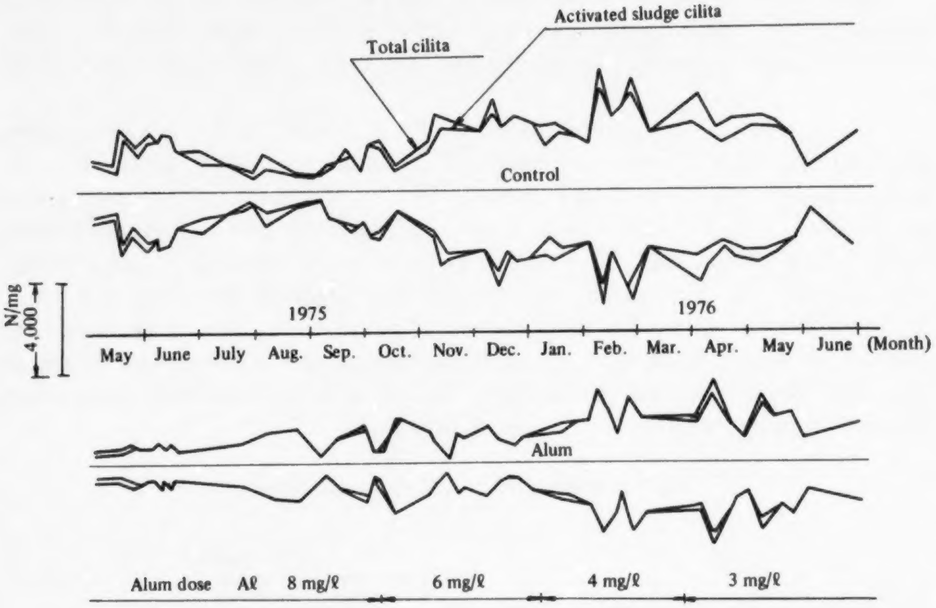
Fig. 1.9 is the comparison between the two bays of the dehydrogenizing activity of activated sludge when 8 mg/ℓ of aluminum was added. It shows hourly changes at different points in the aeration tank. Dt on the figure shows total activity. De shows activity after substrates are washed out (that of eudonegous respiration) On the control side, Dt is always higher than De, while De is higher on the alum dose side. In other words, activity rises after substrates are washed out. There is little difference in the De value between the two bays. This indicates that the addition of 8 mg/ℓ of aluminum is interferring biological activities in one way or the other. Similar phenomena are often observed in sewage treatment plants where relatively large volumes of industrial waste flow in.

Fig. 1.9 Comparison of Dehydrogenaze Activity



As for changes in protozoa, the numbers of spieces and population proved to decline in the alum bay comparing to the control bay when the concentration of added aluminum was high. Fig. 1.10 shows monthly changes in the population of total ciliata and activated sludge ciliata per unit of MLSS in both bays. Although alum addition reduces the population, the dgree differs with the dose rate of aluminum. There is little difference between the two bays when aluminum is below 4 mg/ℓ (below 50 mg/ℓ as alum). Thus, it seems to be possible to assume that there is little practical effect on protozoa when added alum is below 50 mg/ℓ. As mentioned earlier, this dose rate is also effective for phosphorus removal.

Fig. 1.10 Comparison of Ciliata Number in Activated Sludge



Alum addition would considerably reduce the population of swimming type ciliata (difference between total ciliata and activated sludge ciliata). But it is not yet clear – whether the reduction is a direct effect of aluminum or it is a result of the change in environment by advanced removal of organic substances.

1.3.7 MATERIAL BALANCE OF SLUDGE PRODUCTION

It is known that an increase in sludge production is a one of the most troublesome problems in the process. In this plant, continuous monitoring of primary sludge and waste activated sludge is being conducted in both control and alum bays to determine the material balance of sludge production. Sludge production is measured by electro-magnetic flow meters and solids by ultra-sonic sludge density meters.

The increase in sludge production by alum addition is theoretically calculated by the amount of $AlPO_4$ produced by the coagulation and $Al(OH)_3$ by hydrolysis. There is also the transfer of organic sludge as in the control bay.

Table 1.8 is the summary of daily influent loadings and primary sludge in daily average monitored by each phase during the whole experimental period. Shown on the Table is the material balances of sludge production of added aluminum of 8, 6, 4, 3 mgAl/ℓ.

Table 1.8 Comparison of Sludge Production

Phase	II		IV		VI		VII	
Item / Bay	Control	Alum	Control	Alum	Control	Alum	Control	Alum
① Solids in Influent (kg/d)	562		491		491		588	
② Waste Activated Sludge (kg/d)	556	616	374	752	441	695	470	614
③ Solids in Effluent (kg/d)	77	336	140	55	177	152	169	143
④ Total Sludge Production (kg/d) ②+③	633	952	514	807	618	847	639	757
⑤ Concentration*) (mg/ℓ)	53.1	80.0	43.9	68.9	56.7	77.7	48.8	57.9
⑥ Net Sludge Production (kg/d) ④→①	71	390	23	316	127	356	51	169
⑦ Concentration*) (mg/ℓ)	5.9	32.0	2.0	27.0	11.7	32.6	3.9	12.9
⑧ Increased Concentration (mg/ℓ)	26.1		25.0		20.9		9.0	
⑨ Rate of Increase	1.51		1.57		1.37		1.19	
⑩ Alum dose as Aℓ (mg/ℓ)	–	8	–	6	–	4	–	3

*) Divided by Inflow.

The net sludge production which is a difference between total sludge production and solids in influent is also shown in the Table together with a concentration based on the influent flow.

The sludge production in the alum bay increases as the alum dose rate rises. In comparison with the control bay, the sludge production is 1.57 times at 6 mg/ℓ of Al, 1.37 times at 4 mg/ℓ and 1.19 times at 3 mg/ℓ. In terms of net production, the increase rates are much higher. This is because the net sludge production is generally very little in the conventional activated sludge process, in other words, the total sludge production and the solids loadings in the influent are just about the same. Providing that the difference in production rate between the control and alum bays is the amount of increase in inorganic sludge created by alum addition, the increase in concentration is 9.1 ~ 26.9 mg/ℓ. Theoretically, the increase in concentration is about four times that of added aluminum. This corresponds to actual measurements that read 3 ~ 5 times. Therefore, the increase in sludge volume may be estimated by the alum dosing rate. In case the strength of sewage is weak like in our country increase in sludge production ranges from 20% to 60%, more than non-alum process.

As aluminum addition is changed from 8 mg/ℓ to 6, 4, 3 mg/ℓ, the accumulation of aluminum in sludge changes from about 12% to 6, 4.3, 4.0% Al/TS, respectively.

1.4 COSTS OF CHEMICALS

According to the purchase prices in Nagoya City, alum is 21 ¥/kg in terms of 14.5% Al_2O_3 (¥273 per kilogram of aluminum). The chemical cost required is 2.19¥/m^3, 1.64¥/m^3 and 1.09¥/m^3 respectively when the rate of added alum is 100 mg/ℓ, 75 mg/ℓ and 50 mg/ℓ.

Sodium hydroxide is ¥61.3 in terms of one kilogram of NaOH, which means 1.23¥/m^3, 2.45¥/m^3 and 4.9¥/m^3 respectively when the alkalinity addition is 25, 50 and 100 mg/ℓ (as $CaCO_3$).

Aluminum consumed in Japan is almost entirely imported from abroad. Therefore, the cost is rather high compared with other countries and its unit price is unstable. These are the factors that should be taken into account in using aluminum salts as coagulants. The situation is generally the same for iron salts. Therefore, it is strongly considered necessary to recover the used eoagulants from sludges and the projects is now underway at the laboratory experiments.

1.5 SUMMARY

The following has been obtained through a series of experiments in which alum was added to the aeration tank treating typical domestic sewage in Japan, which is relatively weak strength.

1) When phosphorus contained in the influent is below 3 ~ 4 mgT-P/ℓ, an alum addition of 50 mg/ℓ reduces the residual phosphorus to less than 0.5 mg/ℓ.

2) The phosphorus removal capability of return sludge is sufficient enough to be taken into account in controlling alum dose.

3) It is possible to botain nitrified effluent while removing phosphorus. But in the case of a sewage of low alkalinity like one in Japan, it is necessary to add alkalis.

4) Adequate care should be taken in determining the method to dose alkalis to the aerator.

5) The alum addition of 50 mg/ℓ has little practical effect on activated sludge protozoa.

6) The increase in sludge production varies with the alum dose rate, and in this particalr studie increase rate was about 60%.

CHAPTER 2. DEVELOPMENT OF THE DEEP AERATION TANK

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2. DEVELOPMENT OF THE DEEP AERATION TANK

2.1 INTRODUCTION

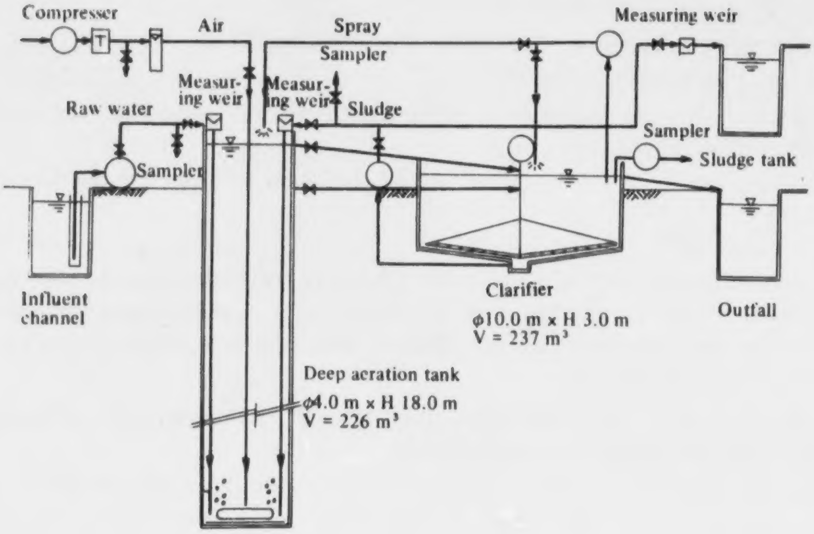
Because of the high prices of land, and the opposition from local inhabitants, it has become almost impossible to purchase new sites for sewage treatment plants within the large cities. And it is proving to be just as difficult to extend facilities at existing plants. Despite this, however, an amount of wastewater and requirements for high purification of wastewater are increasing all the time. The only alternatives left are to go upwards, or go down under the ground. The deep aeration tank is an example of this latter alternative.

A main advantage of increasing the depth of aeration tanks is (1) greater tank capacity within the limited space available. Addition to this (2) higher MLSS concentrations attainable because of the larger saturated concentration value of dissolved oxygen in mixed liquors, and (3) less variation range of dissolved oxygen concentration are expected. (2) and (3) are no more than theoretical predictions at present.

In both Japan and the USA, the standard effective depth of aeration tanks has been 4.5 ~ 5.0 m. The 1975 ASCE Manual No. 36 states that "Ordinarily liquor depths will not be less than 10 feet nor more than 15 feet. The depth is governed in part by foundation and construction costs, in part by the size of the tanks.", while Babbit (1960) adds that "Practice and experience in the United States have led to the adoption of a depth of about 15 ft as representing an economical balance between structural cost and operating cost. A greater depth, to about 20 ft, would give greater efficiency of aeration, but the cost of the tank would be increased, and the higher compression to which the air would be subjected would increase the operating cost. Tanks shallower than about 15 ft would decrease these items but the efficiency of aeration would be uneconomically reduced and the land area required would be proportionally increased." Since then, aeration tanks have always been constructed with depths of 4 to 5 m. It is not exactly clear what happen being associated with the deeper tanks.

The adoption of deeper aeration tanks poses many questions:– (1) would there be sufficient generation of the micro-organisms active for sewage purification, (2) would these micro-organisms settle properly in the final clarifier, and (3) how economical would the running costs of such a deep tank be, etc. In order to answer these and other questions, a pilot plant was set up in 1972, prior to construction of any full scale plants. An outline of this pilot plant is shown in Fig. 2.1.

Fig. 2.1 Schematic Flow Sheet of Pilot Plant



2.2 OVERALL OXYGEN TRANSFER COEFFICIENT VS. POWER CONSUMPTION

The relationships between $K_{La}V$ and water depth, and $K_{La}V$ and air flow rate (based on data from the pilot plant) are shown in Figs. 2.2 and 2.3 respectively.

Fig. 2.2 Depth vs. KLa

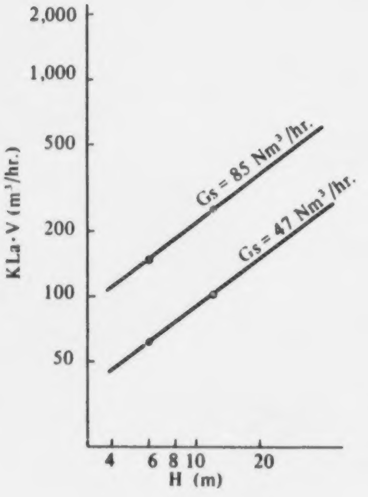
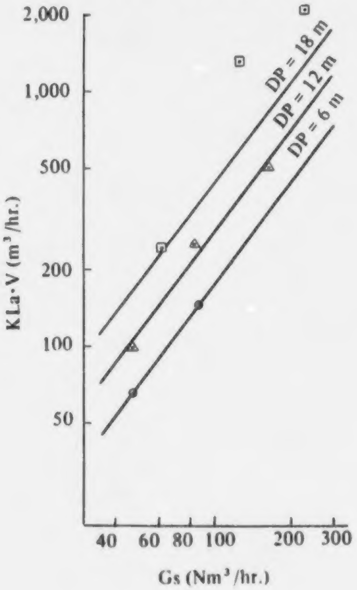


Fig. 2.3 Air Rate vs. KLa·V



Water depth and air flow rate are mathematically related to K_{La} by the following expression,

$$K_{La}V = K'''Gs^{(1-m)}H^{(1-n)} \quad \ldots\ldots\ldots\ldots \quad (1)$$

From the data, equation (1) is changed to equation (2).

$$K_{La}V = 0.085\,Gs^{1.38}H^{0.72} \quad \ldots\ldots\ldots\ldots \quad (2)$$

It is clear that the overall oxygen transfer coefficient will be proportional to water depth raised to the 0.72 power.

The amount of power required for air diffusion is given by the following expression:–

$$L = K \cdot H^{0.67} \quad \ldots\ldots\ldots\ldots \quad (3)$$

A comparison of expressions (2) and (3) above will show that the increase in power requirements due to the deep air diffusion, is more or less offset by the increase in oxygen transfer efficiency. That is, there is little loss of economy caused by the lower diffuser location.

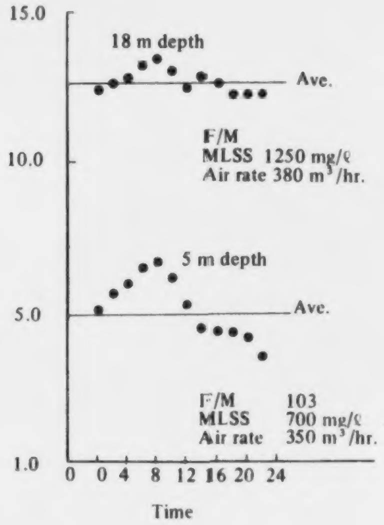
2.3 VARIATION RANGE OF DO CONCENTRATION IN MIXED LIQUOR DUE TO INFLUENT BOD CHANGE

Changes in the influent BOD concentration will result in changes of the oxygen uptake rate (that is, dissolved oxygen concentration) in the mixed liquor. Variation in mixed liquor dissolved oxygen concentration, Δ_c, is related to the oxygen uptake rate, $\Delta\gamma_r$ by the following expression:–

$$\Delta_c = \frac{1}{K_{La}} \cdot \Delta\gamma_r \quad \ldots\ldots\ldots\ldots \quad (4)$$

And since the value of K_{La} is larger in deeper aeration tanks, Δ_c will be smaller for a given $\Delta\gamma_r$. In other words, there should be smaller variations in dissolved oxygen concentration for the deep tanks. In Fig. 2.4, observed values of mixing

Fig. 2.4 DO in Mixed Liquor



liquor dissolved oxygen concentrations are plotted for a 4 m, and an 18 m tank. The amount of variation in the 18 m tank is clearly less (about half) than in the 4 m tank, thus giving proof to the theoretical considerations.

2.4 SEPARATION OF BIO-MASS FROM LIQUID IN FINAL CLARIFIER

Active bio floc was pumped from an existing aeration tank to the 18 m pilot aeration tank. At beginning the concentration of suspended solids in the mixed liquor was observed about 800 mg/ℓ. However, within 20 hours after commencing operation, the MLSS in the pilot aeration tank had fallen to 200 mg/ℓ.

The bio floc had been carried out with the effluent, since it did not without settle in the final clarifier. There was bio floc floating on the surface on the final clarifier (see Fig. 2.6).

Fig. 2.5 MLSS vs. Time

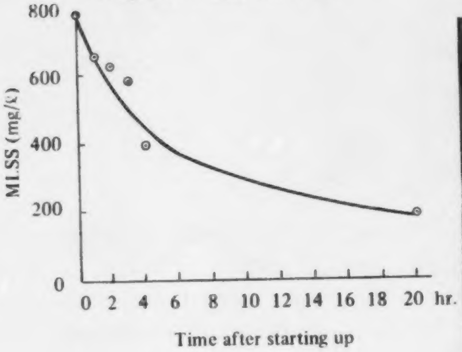
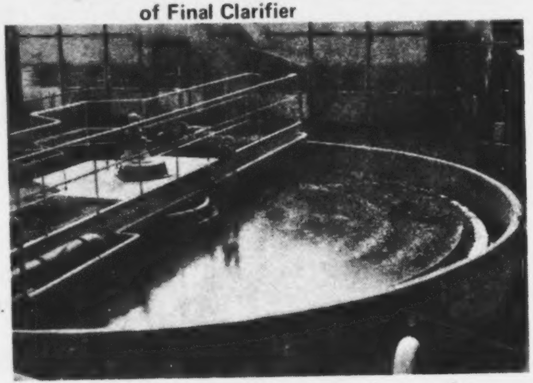


Fig. 2.6 Floated Sludge on the Surface of Final Clarifier



This bio floc accumulated behind the scum baffle, and finally flowed over the weir. But once this problem is solved, the deep aereation tank will be ready for practical use.

Fig. 2.7 shows how a sample of mixed liquor, taken from the above pilot aeration tank, changed over a period of time. The bio floc did not float to the surface immediately after a sample of mixed liquor was poured to the cylinder. Instead, the sludge particles agglomerated together over a certain period of time, and once they reach several millimeters in size, they quickly rose to the top. This rising floc was found to have small air bubbles adhering to it (see Fig. 2.8).

Fig. 2.7 Floating Sludge with Time

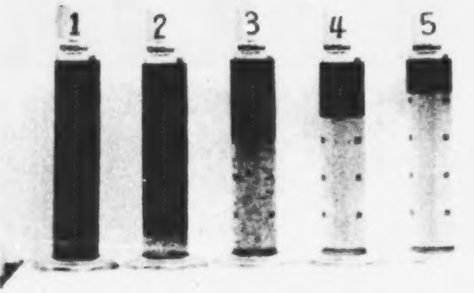
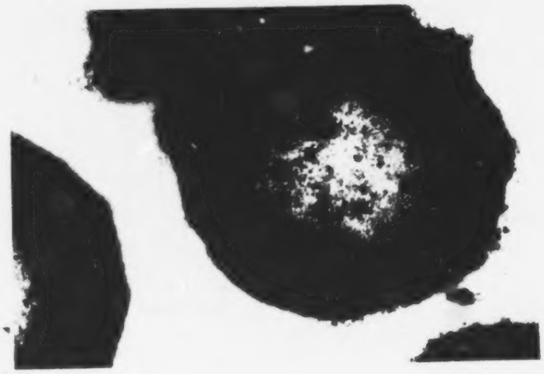


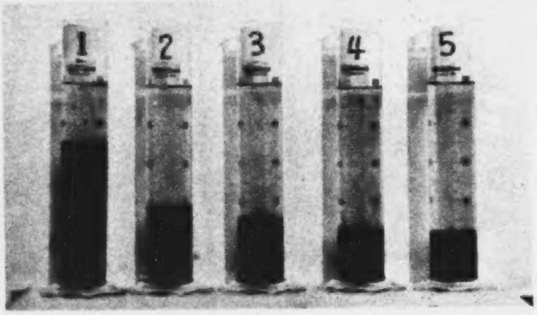
Fig. 2.8 Floc with Adhering Air Bubbles



A special analysis proved that the air bubbles contented a high nitrogen. Nitrogen in the air which was diffused into the mixed liquor at a depth of 18 m dissolved up to saturating the liquor with nitrogen. But since the saturated nitrogen concentration near the surface is lower, the excess dissolved nitrogen reverts to the gaseous state, forming small bubbles of gas.

Next, a sample of mixed liquor was taken from an aeration tank where the diffuser was located at a depth of only 4 m. The floc behaviour was again examined (see Fig. 2.9).

Fig. 2.9 Floc Behaviour (in 4 m Aeration Tank)



In this case, the floc settled very well. This illustrated very clearly the close relation between diffuser position and floc floating/settling behavious.

In order to purge the floc of the adhering bubbles, re-aeration of the mixed liquor was tested. All of the mixed liquor from the 18 m aeration tank was flowed through a series of four 4.5 m re-aeration tanks, and re-aerated for 40 minutes (10 mins. in each tank). Samples were taken from each tank, and allowed to rest for 30 min. The degree of settling in each case has been shown in Fig. 2.11.

Fig. 2.10 Reaeration

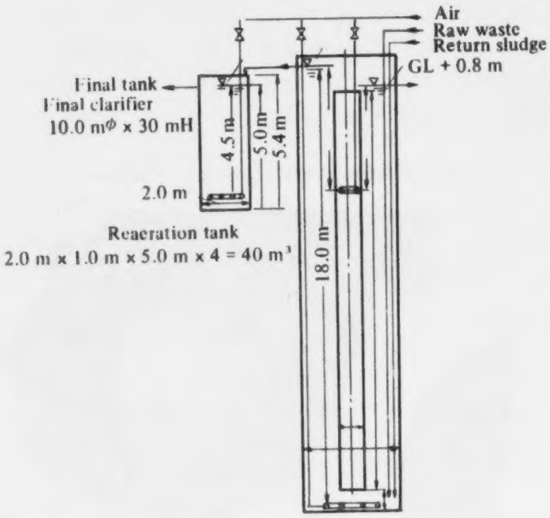
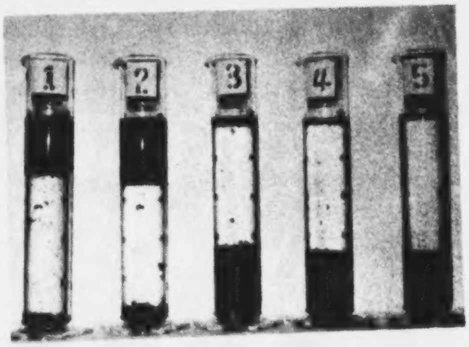


Fig. 2.11 Effects of Reaeration



The results revealed that re-aeration for 20 minutes was sufficient to prevent the floc from rising to the surface. Table 2.1 listed the concentrations of dissolved oxygen, nitrogen, and carbon dioxide in the mixed liquors of each tank. A comparison of this table with Fig. 2.10 indicated that the floc started to rise when the nitrogen concentration exceeded 20 mg/ℓ. Special attention should be paid to the fact that it is possible to prevent floc from rising without reducing the dissolved nitrogen concentration as low as the saturation level (14.9 mg/ℓ) under atmospheric pressure. This threshold concentration of 20 mg/ℓ corresponded to the saturated concentration of dissolved nitrogen at a water depth of 5.8 m. The aeration depth, 5 m, which has traditionally been a standard has a meaning, although they do not understand significance of the meaning. The 5 m are the depth in which bio flocs are free from floating, since disolved nitrogen concentration in the depth is a bit inside the threshold.

Table 2.1 Dissolved Gas Concentration Observed

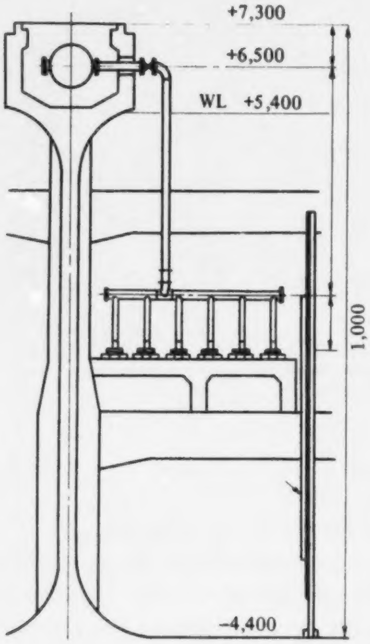
Location	O_2 (mg/ℓ)	N_2 (mg/ℓ)	CO_2 (mg/ℓ)
18 m Depth	12.7	27.9	30.1
	12.0	26.4	31.6
#1 Re-Aeration Tank	8.8	22.3	–
	9.4	22.1	16.0
#2 Re-Aeration Tank	8.3	20.4	–
	8.2	19.8	27.7
#3 Re-Aeration Tank	7.8	19.0	–
	8.6	18.8	23.2
#4 Re-Aeration Tank	8.7	18.9	–
	9.3	19.3	21.0
Standard Aeration Tank	3.2	18.3	44.4

2.5 DESIGN AND OPERATION OF FULL SCALE DEEP AERATION TANK IN TOKYO

2.5.1 DESIGN PARAMETERS

A deep aeration tank has been installed at the Morigasaki Sewage Treatment Plant in Tokyo. Since this area is reclaimed foreshore, the ground is still not very firm, making it difficult to excavate beyond 10 m. Consequently, the effective depth of this tank is 10 m. And in order to prevent floatation of the bio floc, the diffuser has been set half way down the tank. A cross-sectional diagram of the tank is shown in Fig. 2.12.

Fig. 2.12 Cross Section of Full Scale Deep Aeration Tank



The baffle plate running down the center of the tank has been installed to assist the mixing. The design criteria are as follows:

Influent flow rate: 1,860,000 m^3/day
Influent BOD: 150 mg/ℓ
Dimensions of aeration tank: 8.4 m wide, 100 m long, 10 m deep 4 turns/tank
Number of aeration tanks: 12
Volume capacity: 31,000 m^3/tank
Aeration time: 4 hours
Air flow rate: 7.5 times of influent flow rate

2.5.2 DISTRIBUTION OF SUSPENDED SOLIDS AND STREAM VELOCITIES IN THE AERATION TANK

Stream velocities actually measured in aeration tanks where the diffused air flow rates were 5 and 7.5 times as much as the amount of influent are shown in Fig. 2.13.

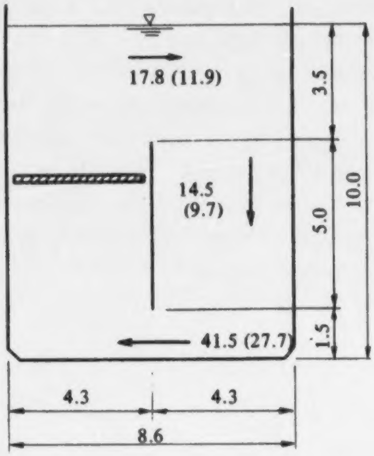


Fig. 13 Velocity of ML in Full Scale Tank

Stream velocities measured 30 to 40 cm/sec by the walls of the tanks, and 20 cm/sec near the baffle plate. There was little change in these figures when the air flow rate was increased.

The MLSS concentration throughout the tank averaged 2,500 mg/ℓ (2,700 mg/ℓ max., and 2,100 mg/ℓ min.), which means that distribution was fairly even. There was no accumulation of bio floc at the bottom or the top of the tank.

2.5.3 PERFORMANCE AND ECONOMICAL FEASIBILITY

Influent BOD:	100 mg/ℓ
Effluent BOD:	4.5 mg/ℓ
Influent temperature:	20°C
Detention (for Q):	4.6 hrs.
F/M:	0.2 kg/BOD/kgSS/day
BOD volume loading:	0.53 kg/m^3/day
Air flow rate:	3.3 times
Power consumption:	0.81 kWH/g. BOD 0.081 kWH/m^3 (0.083 kWH/m^3 at existing plant)

These figures show that the power consumption (kWH per m^3) required to operate deep aeration tanks is much the same as that required for standard depth aeration tanks.

2.5.4 CONSTRUCTION COSTS

The construction cost for unit volume of the 10 m deep aeration tank is approximately 1.4 times higher than a 5 m deep tank.

2.6 COMPARISON OF DIFFUSER AND JET AERATOR IN DEEP AERATION TANK

In recent years, the jet aerator developed in the states has also been available in Japan. It is well known that although the running cost is high, it features a greater oxygen transfer efficiency. This means that it can transfer greater amounts of oxygen without increasing the air flow rate.

In recent years, the jet aerator developed in the States has also been available suddenly increased because of several reasons after the blower room and main air pipes had been completed. There was no available space left to extend facilities to cope with the increased quantities, so the only alternatives were to deepen the aeration tanks, and improve the oxygen transfer efficiency without increasing air supply equipment. The jet aerator seemed to answer the needs very well.

The aeration tanks at this plant measure 10 m deep, 10 m wide, and 76 m long. The jet aerator was installed at 5 m below the surface. Performance of the system were as follows.

Influent BOD: 110 mg/ℓ
Effluent BOD: 10 mg/ℓ
Detention time (for Q): 7 hrs.
F/M: 0.3 kg BOD/kgSS/day
Air flow rate: 0.37 kg BOD/m^3
Power consumption: 2.9 times as much as influent rate

	kHW/kg BOD	kHW/m^3/inflow
Pump		
Blower		
Total		

Fig. 3.14 Full Scale Deep Aeration Tank in Hiagari

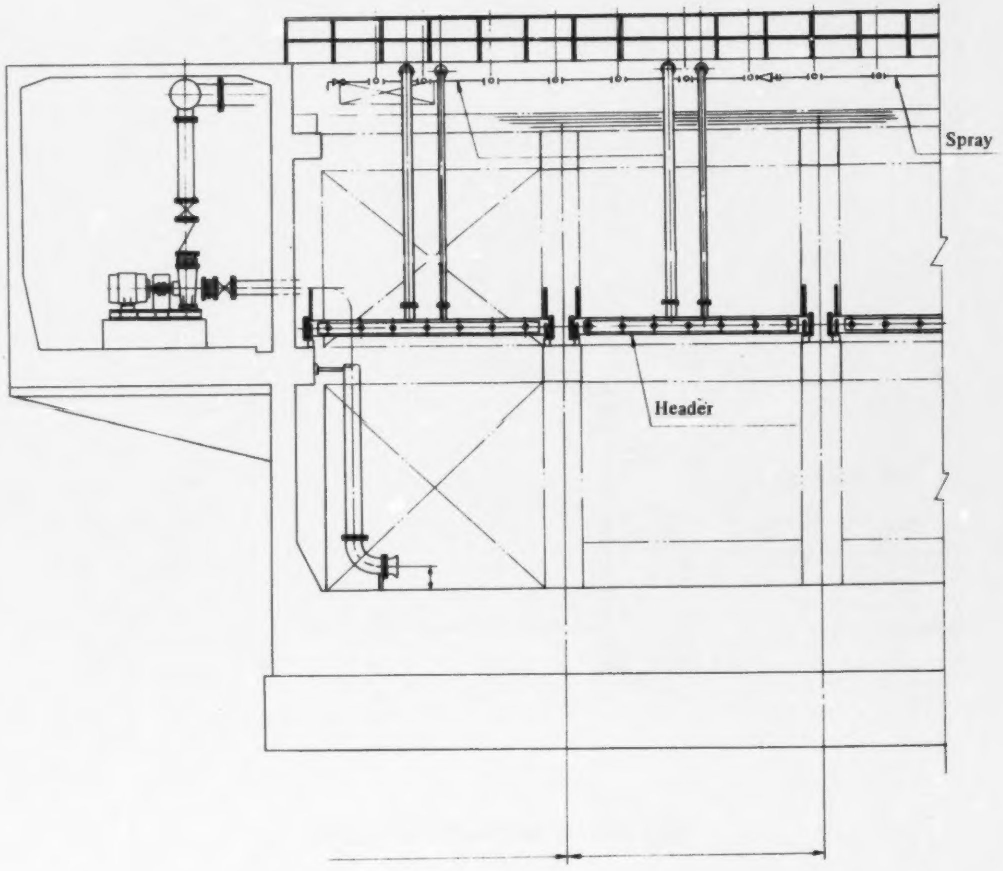
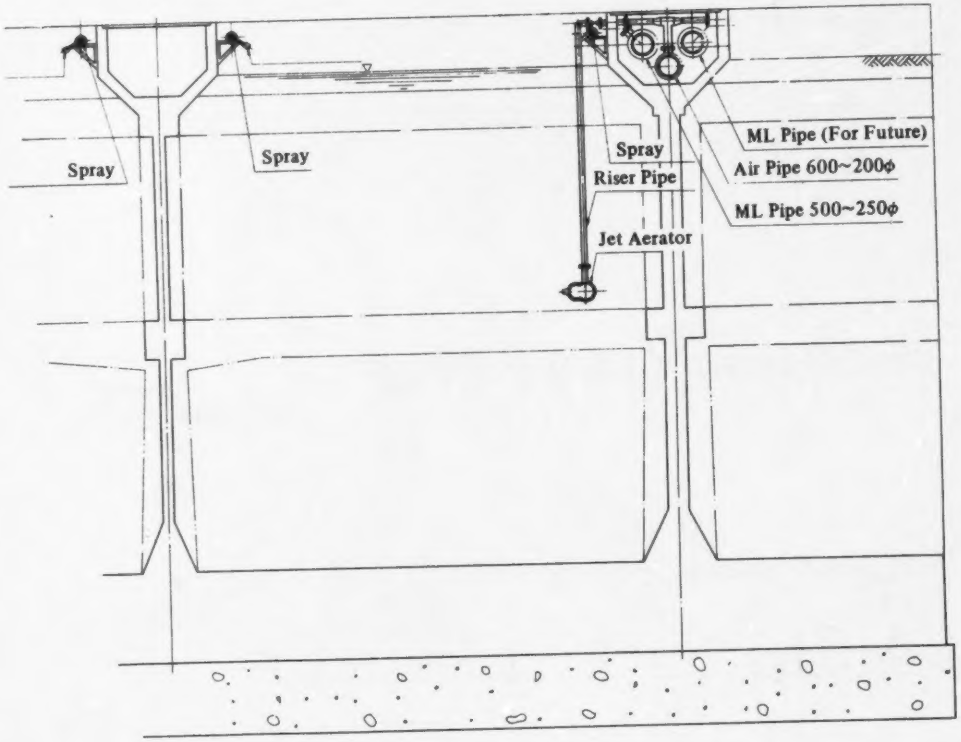


Fig. 3.15 Full Scale Deep Aeration Tank



CHAPTER 3. EXPERIMENTAL STUDIES ON PERFORMANCE OF RAPID SAND FILTRATION PROCESS FOR TERTIARY PURPOSE

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3. EXPERIMENTAL STUDIES ON PERFORMANCE OF RAPID SAND FILTRATION PROCESS FOR TERTIARY PURPOSE

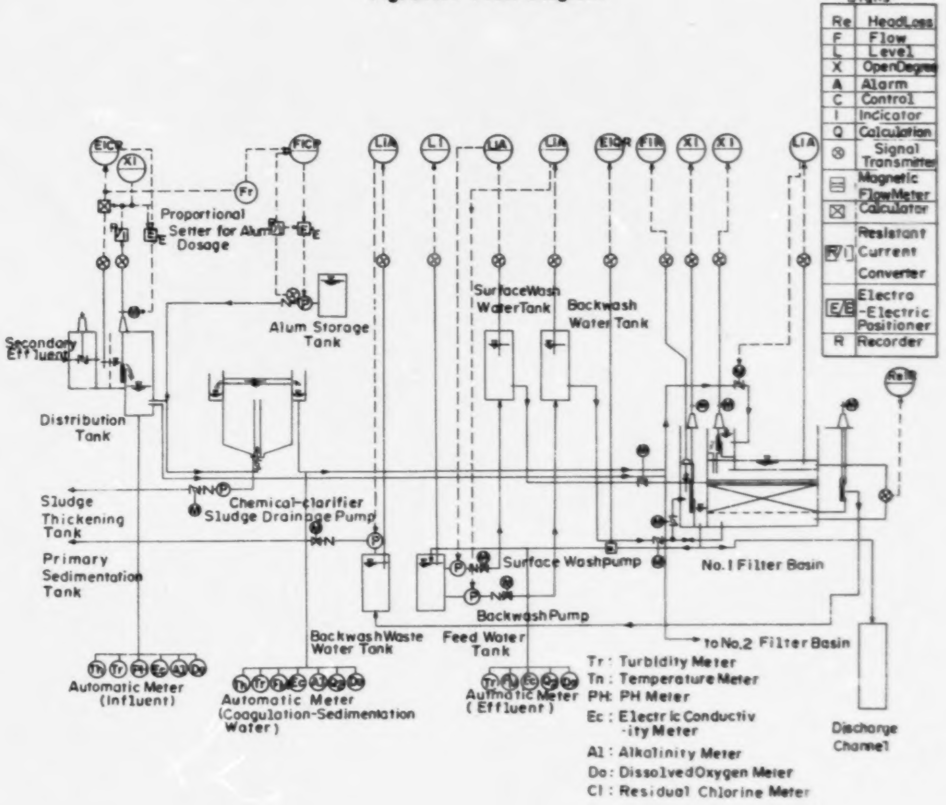
3.1 FILTRATION STUDY AT THE TOKYO FULL SCALE PLANTS

3.1.1 COAGULATION-SEDIMENTATION FILTRATION EXPERIMENT

a. Generality of the Tertiary Treatment Plant

The department of Sewage Works of Tokyo Metropolitan Government and the Ministry of Construction have jointly constructed the pilot plant of practical scale for tertiary treatment in Minamitama Sewage Treatment Plant of Tokyo. The construction started in August of 1974 and was accomplished in March of 1976. This plant comprises a sedimentation tank and two filter basins for the purpose of removing suspended solids, BOD and phosphorus from the secondary effluent of sewage treated by the activated sludge method.

Fig. 3.1.1 Flow Diagram



Concerning the treatment capacity of the plant, the coagulation-sedimentation capacity is 8,800 m^3/day (2.32 MGD) and the filtration capacity is 17,600 m^3/day (4.65 MGD).

This plant is constructed so that the flow may be divided into two for the experimental purpose as shown in Fig. 3.1.1; one of the flows is that of filtration to directly filter the secondary effluent of sewage and another is that of coagulation-sedimentation filtration to filter the secondary effluent of sewage after its coagulation and sedimentation. The coagulation-sedimentation tank is the type of sludge circulation, which is the rectangular tank with the surface area of 225 m^3 (2,421 SF) and a side of 15 m (49.2 FT) made from the reinforced concrete.

The upflow speed of the effluent of sewage is 29.3 mm/min. (1.15 inches/min.) based on the average treatment capacity (8,800 m^3/day) and the average staying time in this tank is 22.36 h. based on the average treatment capacity. The rotation speed of the impeller is 2.65 r.p.m., that of the scraper 3.9 r.p.h. and the width of the impeller band 50 mm (2 inches). The surface area of each filter basin is 57.8 m^2 (622 SF) its depth is 8.40 m (27.6 FT), the thickness of the filter layer is 1.20 m (4 FT) and the excess height is 5.90 m (19 FT). This is non-constant-pressure filter basin whose water level rises as the head loss increases.

Influent into the filter basin can be controlled constantly with the distribution tank. The design filter speed is 152.2 m/day (2.59 GPM/SF). The structure of the filter layer is shown in Table 3.1.1. The wastewater produced by the cleaning of the filter basin shall be stored once in the cleaning wastewater tank and then sent to the primary sedimentation tank of the sewage treatment plant. The treatment capacity of this plant is below that of the tertiary treatment so that it is made a rule to perform the tertiary treatment of all the secondary effluent of sewage.

Table 3.1.1 Structure of Filter Media

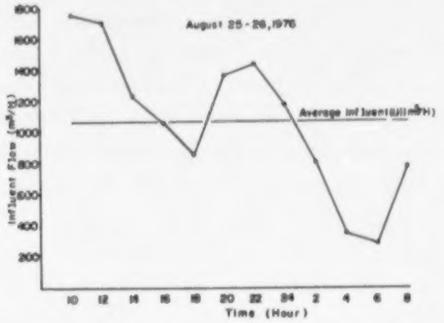
Filter Media	NO. of Filter Basin	Thickness of Layer (mm)(inch)	Effective Size(mm)	Uniformity Coefficient
Sand	NO.1 Filter Basin	625(24.6)	0.61	1.39
	NO.2 Filter Basin	625(24.6)	0.71	1.39
Anthrocite	NO.3 Filter Basin	375(14.8)	1.35	1.39
	NO.2 Filter Basin	375(14.8)	1.52	1.38
Supporting Gravel	NO.1,NO.2 Common	50(2.0)	2.00-3.36	—
	″	50(2.0)	3.36-6.73	—
	″	50(2.0)	6.73-12.7	—
	″	50(2.0)	12.7-19.1	—

b. Generality of the Secondary Treatment Plant

This sewage treatment plant has only Tama-Newtown (the number of families 13,571 and the population 46,160 as of 1976, Nov. 1) as its drainage district and it collects sewage through the sewer of separate system so that its maximum sewage amount per hour is almost six times of its minimum sewage amount per hour as shown in Fig. 3.1.2. Accordingly, the maximum sewage amount per hour is 1.58 times of the average sewage amount of a day and the minimum sewage amount per hour is 0.27 times of the average sewage amount of a day.

The peak time of the entering sewage amount is 10:00 a.m. and 10:00 p.m., and the entering sewage amount becomes extremely low at 6:00 a.m. For this reason this sewage treatment plant is difficult to control the operation.

Fig. 3.1.2 Hourly Variation of Influent Amount



The operating condition of the secondary sewage treatment facility in this plant is indicated in Table 3.1.2. The hourly variation of the quality of the secondary effluent of this plant compared to the raw sewage is as shown in Fig. 3.1.3 ~ 3.1.5. The concentration of suspended solids (SS) in the raw sewage attains to the peak both at 10:00 a.m. and 10:00 p.m. and this coincides with the peak hour of the entering sewage amount. At the peak time of 10:00 a.m., the SS concentration of the raw sewage was almost 425 mg/ℓ (3.8 times of the average SS concentration of the raw sewage) and it was found that this concentration was quite larger than that of almost 160 mg/ℓ (1.4 times of the average SS concentration of the raw sewage) at the peak hour of 10:00 p.m. The SS concentration of the secondary effluent became the highest; almost five times of the average SS concentration of the effluent, influenced by the peak of the entering amount. The total phosphorus (T-P) concentration of the raw sewage attained to the peak at 10:00 a.m. and it was almost 2.7 times of the average T-P concentration. The T-P concentration of the secondary effluent was averaged by the secondary treatment and was constant with almost no hourly variation.

The ammonia nitrogen concentration in the raw sewage attained to the peak also at 10:00 a.m. (2.3 times of the average concentration) and there was almost no variation of the effluent found. These continuous water quality tests during 24 hours were taken place 3 times in August. The load quantity, the loading average concentration and the removal rate concerning every waste water quality were determined based on the wastewater quantities and the concentrations obtained during the hourly measurement, and the average value of the three tests are shown in Table 3.1.3. The wastewater load per person in this sewage treatment district was found to be 374ℓ (99 gallon) and BOD load to be 68 g (0.15 lb.), based on the numerical value of 1976, Nov. 24 shown in Table 3.1.1. The other loads per person are as shown in Table 3.1.4, being determined upon the Table 3.1.3.

Table 3.1.2 Operational Conditions of the Secondary Treatment Facilities of Minami-Tama Sewage Treatment Plant

Year and Date / Items	1975			1976			
	5.14	8.27	11.13	2.17	5.11	8.12	11.24
Effluent Quantity (m³/day)	9510	10 300	9240	9 980	13880	14670	17250
Returned Sludge Ratio (%)	100	94	104	94	79	87	86
Average Aeration Time (hour)	77	73	78	76	59	53	48
Average Detention Time (Primary Sedimention) (hour)	*1 49	60	67	62	45	42	*2 72
Average Detention Time (Secondary Sedimention) (hour)	45	43	45	44	3.4	3.1	28
Air Blow Ratio (Air Blow Quantity / Treated Water)	79	64	62	69	60	88	80
Sludge Age (day)	128	253	181	66	58	64	4.2
BOD Capacity Load (Kg/m3/day)	022	0.17	0.17	0.20	031	029	0.44
BOD Load (Kg/MLSS Kg/day)	0.16	0.09	0.10	0.20	0.26	0.15	0.37
Average BOD of Sewage Flowing into Aeration Tank *3 (mg/l)	143	98	110	120	137	120	157
Average SS of Sewage Flowing into Aeration Tank *3 (mg/l)	71	42	60	92	92	122	100
MLSS *4 (mg/l)	1419	1795	1649	995	1205	1892	1187
MLVSS *4 (mg/l)	994	919	1060	857	938	1157	899
MLVSS / MLSS (%)	70	51	65	86	78	61	76
SV *4 (%)	22	21	28	80	38	28	26
SVI	152	115	167	802	312	148	215

Effluent Quality	PH	Influent	72	73	73	73	71	71	73
		Effluent	71	70	69	67	70	6.9	4
	Transparency (cm)	Influent	5	7	35	5.0	6	6.5	5
		Effluent	85	100	100	100	100	50	64
	CODMn (mg/l)	Influent	78	68	64	72	62	49	64
		Effluent	12	8	8	9	8	8	10
	SS (mg/l)	Influent	138	116	130	126	118	102	128
		Effluent	6	3	5	2	4	8	5
	BOD (mg/l)	Influent	178	161	185	160	180	113	183
		Effluent	5	3	4	2	8	6	4

*1 Primary Sedimentation Capacity, 1950m³

*2 " " " 5180m³ (183,000CF)

*3 Composite sample

*4 Average value at 9 o'clock and at 12 o'clock at the end of the Aeration tank.

Table 3.1.3 Removal Ratio by Secondary Treatment

Section / Water Quality Items	Loading		Average Concentration		
	Influent Load Capacity Kg/day (LBS/day)	Effluent Load Capacity Kg/day (LBS/day)	Influent (mg/l)	Effluent (mg/l)	Removal (%)
S S	2,021 (4,456)	150 (331)	152	11	93
T - P	70 (154)	41 (90)	53	2.7	41
N H₄ - N	307 (677)	17 (37)	233	1.3	94

Note; Measuring dates are three times of Aug. 11-12, Aug. 18-19 and Aug. 25-26 in 1976

; Removal can be calculated out from the load capacity

Fig. 3.1.3 Hourly Variation of SS Concentration in Secondary Effluent

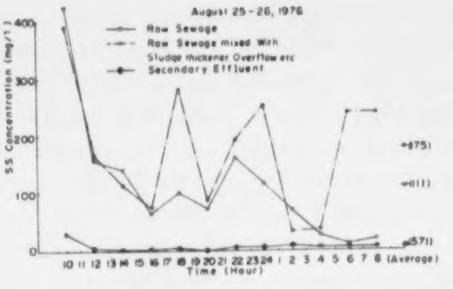


Fig. 3.1.4 Hourly Variation of T-P Concentration in the Secondary Effluent

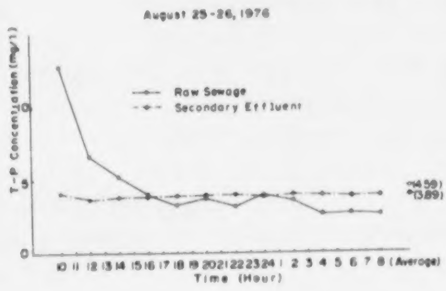


Fig. 3.1.5 Hourly Variation of NH⁺, -N Concentration in the Secondary Effluent

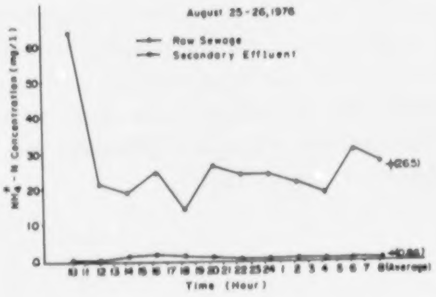


Table 3.1.4 Pollution Load Per Capita

Water Quality Items	Pollution Loads Per Capital g/day/person (LBS/day/100 persons)
S S	4.4 (0.97)
T - P	1.5 (0.33)
N H₄ - N	6.7 (1.48)

c. The Experimental Result of the Tertiary Treatment

The experiment of the tertiary treatment was conducted, using alum ($Al_2(SO_4)_3 \cdot 18H_2O$) as the coagulant. The alum was added automatically in proportion to the influent amount and the added amount during each experimental period is as shown in Table 3.1.5. The experimental result of the tertiary treatment is indicated in Table 3.1.6. When alum was added below 40 mg/ℓ, the formation of flocs was insufficient and subsequently the formation of sludge blanket in the coagulation-sedimentation tank becomes insufficient. Particularly when the dosage was 30 mg/ℓ, no blanket was observed. In the case of 30 ~ 40 mg/ℓ of alum dosage the SS removal was higher in the directly filtered effluent than in the coagulation filtered effluent as shown in Fig. 3.1.6. As for the coagulation-sedimented wastewater, the average removal was extremely low; below 25%. This was caused by carry-over of the floc formed in the coagulation-sedimentation tank. The BOD removal in this case was not greatly different with coagulation-sedimentation filtration and with direct filtration; it was around 75% as shown in Fig. 3.1.7. The coagulation-sedimented wastewater, however, as the flocs caused carry-over and intermixed into the effluent, had very low BOD removal. The T-P removal in this case was around 60% as shown in Fig. 3.1.8 in the coagulation filtered water indicating incomplete coagulation reaction and moreover around 45% in the coagulation-sedimented water due to carry-over, both being low removal. But it was largely different from the removal of 7% of the direct filtration and this proved the reaction effect of the coagulant. When raising the dosage to 50 ~ 60 mg/ℓ, the coagulation reaction was almost completely performed and the produced flocs formed the blanket zone in the coagulation-sedimentation tank. For this reason, the T-P removal of the coagulation-sedimentation filtered water attained to the peak of 95%. Also concerning the coagulation-sedimented wastewater, T-P removal was 90% at the dosage of 60 mg/ℓ, not so less than the coagulation-sedimentation filtered effluent. But concerning SS, as indicated in Fig. 3.1.6, the removal of the coagulation-sedimentation filtered effluent was only 5% above that of the direct filtered effluent. The removal of the coagulation-sedimented effluent was also not more than 50% and it showed that outflow of fine flocs was inevitable due to the inflow fluctuation. Also as to the BOD, the difference of the BOD removal between the coagulation-sedimentation filtration and the direct filtration was 5%; degree of the coagulation effect being slightly acknowledged.

Table 3.1.5 Alum Dosage

Experiment Number	Experiment Period	Design Alum Dosage (in the effluent)	Average Alum Dosage (Al/P Mol Ratio)
I	From Sep.20 to Oct.14 1976	60 mg/l	2.1
II	From Oct.16 to Nov.18 1976	50 "	1.6
III	From Dec 20, 1976 to Jan 10, 1977	40 "	1.1
IV	From Dec 3 to Dec 18 1976	30	0.9

Table 3.1.6 Experiment Results

Item			Experiment Period	I				II				III				IV			
			Alum Dosage	60ppm				50ppm				40ppm				30ppm			
	Division			N	Mx	Mi	Av	N	Mx	Mi	Av	N	Mx	Mi	Av	N	Mx	Mi	Av
COD_{Mn}	Influent	Total	Concen-tration	20	7.6	5.4	6.4	20	18.3	5.6	[illegible]	12	17.5	9.0	13.5	14	18.7	[illegible]	13.6
		Dead	Concen-tration					20	[illegible]	3.4	7.1	12	15.6	[illegible]	10.8	12	14.8	[illegible]	12
			D/T						87	40	77		[illegible]	67	79		93	78	80
	Coagulation -Sedimentation		Concen-tration	20	[illegible]	3.6	4.5	20	7.8	4.2	6.0	12	16.9	[illegible]	11.4	14	14.3	8.2	11.2
			Removal	20	41	27	33	20	67	16	33	12	28	34	16	14	26	13	16
	Direct Filtration		Concen-tration	20	6.0	3.5	4.8	20	[illegible]	4.4	6.3	12	19.9	6.4	11.1	14	12.6	7.8	9.8
			Removal	20	51	13	25	20	63	20	30	12	36	0	22	14	34	20	27
	Coagulation -Sedimentation Filtration		Concen-tration	20	5.2	3.3	3.9	20	6.6	4.1	5.2	12	15.8	5.8	[illegible]	14	11.4	6.1	[illegible]
			Removal	20	53	65	40	20	71	30	43	12	41	9	32	14	46	26	34
T - P	Influent		Concen-tration	16	3.5	2.1	2.6	20	4.09	2.65	3.26	12	3.79	3.08	3.37	14	4.16	2.95	3.34
	Coagulation -Sedimentation		Concen-tration	16	0.67	0.16	0.37	20	1.22	0.61	1.02	12	2.27	1.36	1.97	14	2.41	1.02	1.97
			Removal	16	[illegible]	78	86	20	78	59	69	12	56	35	40	14	66	30	40
	Direct Filtration		Concen-tration	16	3.40	1.98	2.6	20	3.94	2.46	2.99	12	3.49	2.87	3.28	14	3.60	2.79	3.13
			Removal	16	31	40	79	20	27	10	8	12	11	6	8	14	10	0	6
	Coagulation -Sedimentation Filtration		Concen-tration	16	0.36	0.05	0.12	20	0.96	0.30	0.61	12	1.56	0.77	1.38	14	1.86	0.79	1.46
			Removal	16	96	94	96	20	89	71	81	12	75	56	62	14	74	48	56
Al/P Ratio	Ratio to Total		T - P	1.995				1.428				1.043				0.841			
	Ratio to Soluble		T - P	2.146				1.557				1.135				0.892			

Table 3.1.6 Continued

Item			Experiment Period	I				II				III				IV			
			Alum Injection Rate	60ppm				50ppm				40ppm				30ppm			
	Division			N	Mx	Mi	Av	N	Mx	Mi	Av	N	Mx	Mi	Av	N	Mx	Mi	Av
S.S	Raw Water		Concen-tration	16	8.2	2.1	[illegible]	20	34.4	3.4	[illegible]	12	18.3	7.0	12.0	14	14.0	4.0	[illegible]
	Coagulation -Sedimentation		Concen-tration	16	10.1	0.2	3.0	20	[illegible]	1.7	3.2	12	14.8	5.7	[illegible]	14	13.0	2.5	[illegible]
			Removal	16	97	0	45	20	[illegible]	0	52	12	64	0	23	14	70	0	17
	Direct Filtration		Concen-tration	16	1.5	0.1	[illegible]	20	[illegible]	0.1	0.4	12	4.1	[illegible]	2.0	14	2.6	0.4	1.2
			Removal	16	98	71	91	20	99	[illegible]	94	12	[illegible]	[illegible]	84	14	[illegible]	[illegible]	[illegible]
	Coagulation -Sedimentation Filtration		Concen-tration	16	[illegible]	0	0.2	20	0.5	0	0.2	12	5.1	1	2.5	14	4.1	0.1	1.9
			Removal	16	100	71.4	95	20	100	93	[illegible]	12	87	58	79	14	98	42	75
Turbidity	Raw Water		Concen-tration	11	[illegible]	3.5	8.0	20	24.2	2.5	6.3	12	111	7.8	24.0	14	14.1	4.5	[illegible]
	Coagulation -Sedimentation		Concen-tration	11	[illegible]	0.4	0.8	20	3.3	0.5	1.1	12	26.1	2.6	7.9	14	4.4	0.2	2.3
			Removal	11	94	82	87	20	96	42	76	12	77	30	58	14	[illegible]	[illegible]	87
	Direct Filtration		Concen-tration	11	0.6	0.2	0.3	20	1.0	0.2	0.6	12	31.3	1.2	6.2	14	3.3	0.3	1.4
			Removal	11	98	85	90	20	96	82	90	12	88	67	80	14	97	81	90
	Coagulation -Sedimentation Filtraton		Concen-tration	11	0.14	0	0.06	20	0.4	0	0.1	12	22.8	0.3	4.2	14	1.6	0.07	[illegible]
			Removal	11	100	[illegible]	92	20	100	93	[illegible]	12	96	78	[illegible]	14	98	76	[illegible]
BOD	Raw Water	Total	Concen-tration	5	5.9	3.0	4.3	10	16.0	4.9	8.3	4	14.9	10.6	12.8	4	14.6	9.1	12.0
		Dissolved	Concen-tration	5	1.9	0.8	1.3	10	1.9	0.8	1.4	4	3.2	1.3	2.4	4	7.1	1.3	3.3
			D/T		47	18	31		29	9	17		22	14	19		49	12	27
	Coagulation -Sedimented		Concen-tration	5	2.8	0.02	0.9	10	5.2	[illegible]	2.6	4	7.0	3.8	5.6	4	8.2	6.0	7.3
			Removal	5	99	53	83	10	82	37	68	4	73	34	56	4	53	10	36
	Direct Filtration		Concen-tration	5	2.1	0.3	0.8	10	1.8	0.8	1.3	4	6.4	1.7	3.8	4	[illegible]	1.4	3.6
			Removal	5	91	64	83	10	91	73	84	4	87	44	70	4	[illegible]	36	70
	Coagulation -Sedimentation Filtration		Concen-tration	5	1.5	0.2	0.8	10	1.7	0.3	1.0	4	6.6	1.8	3.6	4	4.5	1.1	2.5
			Removal	5	95	76	86	10	94	73	80	4	86	42	74	4	91	70	79

* Unit Concentration ; mg/l
removal ; %

** N ; Number of Samples
Mx ; Maximum
Mi ; Minimum
Av ; Average

Fig. 3.1.6 Comparison of SS Removal

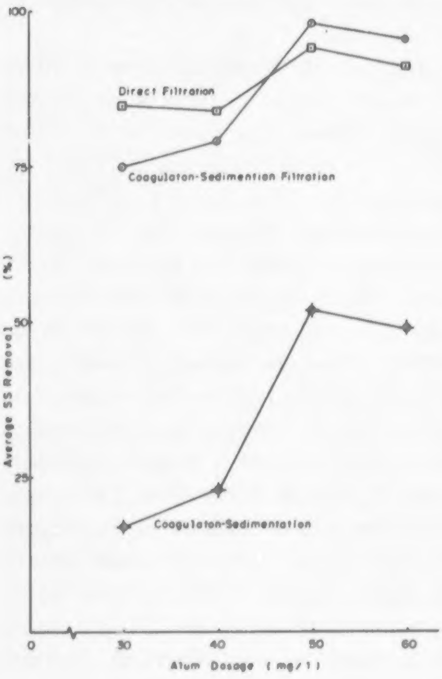


Fig. 3.1.7 Comparison of BOD Removal

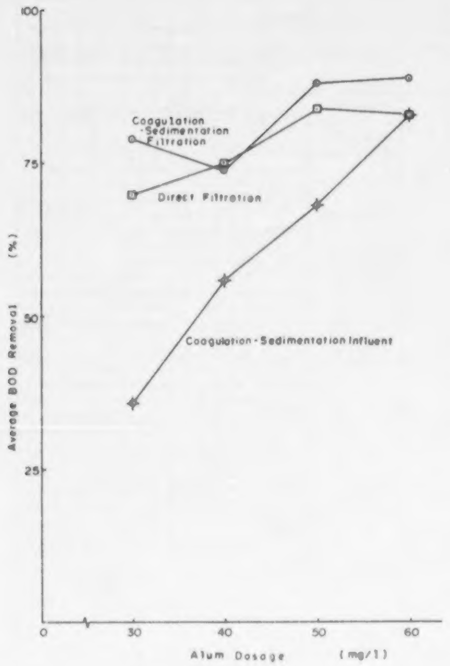
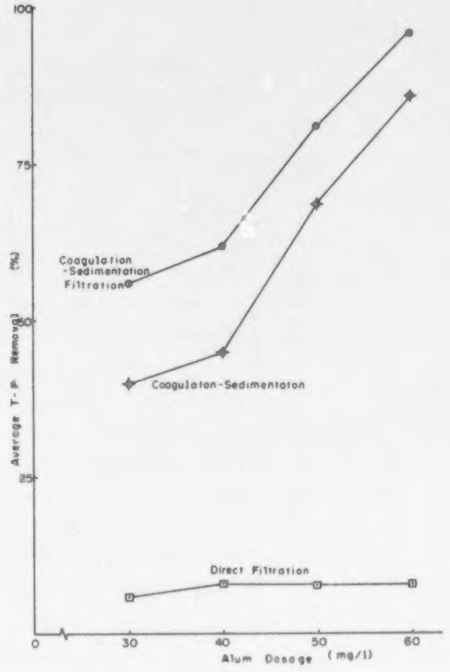


Fig. 3.1.8 Comparison of T-P Removal



The BOD removal, however, became high in the coagulation-sedimentation and as high as in the direct filtration. We consider from this fact that the outflow of fine flocs does not greatly influence the BOD removal.

The data of Table 3.1.1 are obtained from the daily grab samples of 10:00 every morning. In order to investigate the hourly variation of the water quality during continuous 24 hours as for the secondary effluent, the direct filtered effluent, and the coagulation-sedimentation filtered effluent is shown in Fig. 3.1.9 through 3.1.12. The measurement period was from Oct. 13 to Oct. 14, 1976, which were the final dates of the experiment I (the alum dosage; 60 mg/ℓ, the average mol ratio of Al/P; 2:1). Fig. 3.1.9 shows that rapid hourly variation of T-P in the secondary sedimented effluent between 3 ~ 4 mg/ℓ. But in the direct filtered effluent, soluble T-P had not so wide variation between 2.7 ~ 3.3 mg/ℓ. This was due to the fact it was not influenced by the flocs outflown from the secondary sedimented effluent. The addition of alum this day (Al/P mol ratio) is 1.88 on the average, 2.08 in maximum and 1.68 in minimum. This is lower than the average alum mol ratio of 2.1 in the experiment I. Therefore the average T-P concentration in the coagulation-sedimentation filtered effluent of this day was 0.28 mg/ℓ; higher than the average T-P concentration in the coagulation-sedimentation filtered effluent in the experiment I of 0.12 mg/ℓ. But, as the coagulation tank being the type of sludge circulation, when the Al/P ratio varied hourly or in its purposes due to the variation of the T-P concentration in the raw wastewater (secondary effluent), the T-P concentration in the coagulation-sedimentation filtered effluent was not influenced. In other words, when the addition of the coagulant in coagulation-sedimentation treatment was conducted not by the Al/P ratio control but by the flow rate control, we could obtain quite stable phosphorus removal.

Fig. 3.1.9 Hourly Variation of Total Phosphorus Concentration

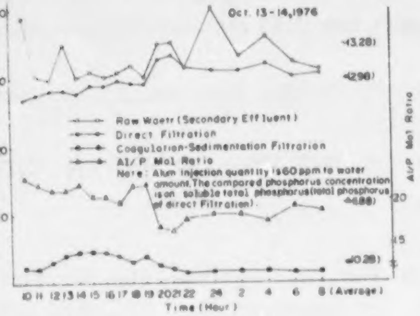


Fig. 3.1.10 Hourly Variation of SS Concentration

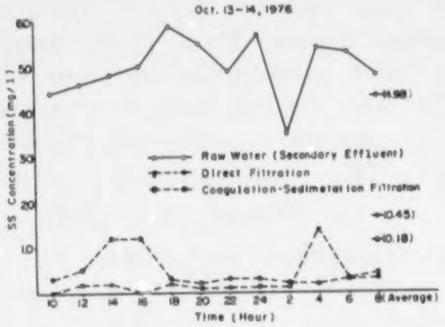


Fig. 3.1.11 Hourly Variation of Turbidity

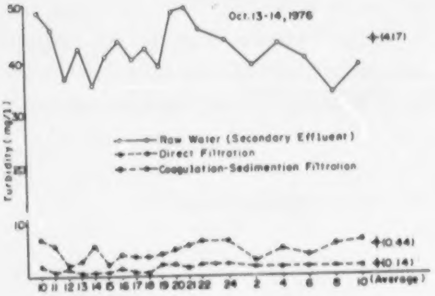


Fig. 3.1.12 Hourly Variation of COD_{Mn} Concentration

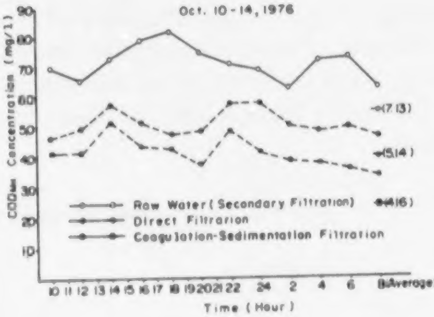


Fig. 3.1.10, 3.1.11 and 3.1.12 show that concerning SS, turbidity and COD_{Mn} the coagulation-sedimentation filtered effluent had a little more excellent water quality than the direct filtered water.

We can get the following summary from the above described;

(1) When dosage of alum is maintained more than 50 mg/ℓ (Al/P mol ratio; 1.6 ~ 2.1), the coagulation reaction would be complete.

(2) Concerning BOD and SS removal rate; there will be no significant difference between the direct filtration and the coagulation-sedimentation filtration: the removals of 70 ~ 85% and 85 ~ 95% respectively were obtained. However, at the dosage of more than 50 mg/ℓ, the removal of the coagulation-sedimentation filtration is 5% more than that of the direct filtration.

(3) Concerning the removal of the phosphorus; the coagulation-sedimentation filtration can provide the removal of around 55% at the alum dosage of 30 mg/ℓ and that of high as 95% at the dosage of 60 mg/ℓ.

(4) Only by coagulation and sedimentation, the stable water quality is difficult to be obtained. Especially at the low dosage of alum the removal of BOD and SS was not improved due to carry-over of flocs. Even at the high dosage, the SS removal rate was not more than 50% because of the outflow of fine flocs and the removal of phosphorus was also 5 ~ 15% less than that of the coagulation-sedimentatio filtered effluent. But at the high dosage of alum, the BOD removal nearly as high as that of the direct filtration will be obtained.

(5) When the addition of alum was conducted by the flow ratio control, the considerably stable removal of phosphorus was obtained.

3.1.2 THE COMPARATIVE EXPERIMENT OF THE UPFLOW FILTER AND THE DOWNFLOW FILTER

a. Generality of the Facilities

With the purpose of comparing the upflow filter and downflow filter, the filter basins were constructed in Morigasaki West Sewage Treatment Plant of Tokyo and the experiment was performed. Its filtration capacity is 24,000 m^3/day (6.3 MGD). The constructed filter basins are two upflow filter basins and two tri-media filter basins as downflow filter. The construction was completed in March, 1974. The flow of the experimental facilities is so that the secondary effluent may be sent simultaneously to the upflow filter basin and to the tri-media filter basin as shown in Fig. 3.1.13. The surface area of both filter basins is 30 m^2 (323 SF) and the structures of each filter basins are shown in Fig. 3.1.14 and Fig. 3.1.15 respectively. The compositions of filter media of each filter basin are indicated in Table 3.1.7 and in Table 3.1.8. The design filter rate is 200 m/day (3.41 GPM/SF) in both filters.

Fig. 3.1.13 Flow Diagram of the Upflow and Tri-Media Filters

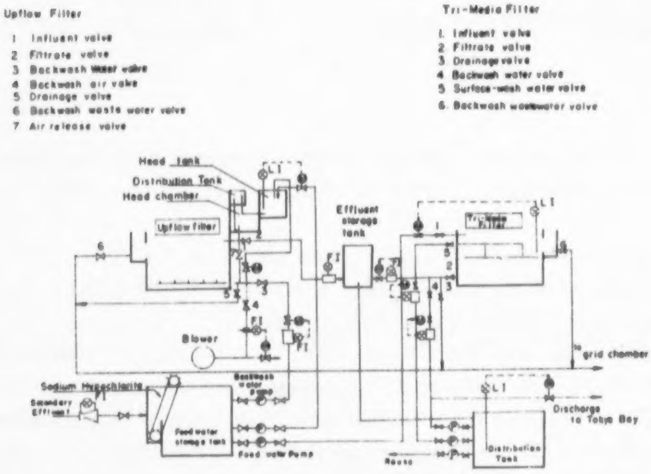


Fig. 3.1.14 Cross Section of the Upflow Filter

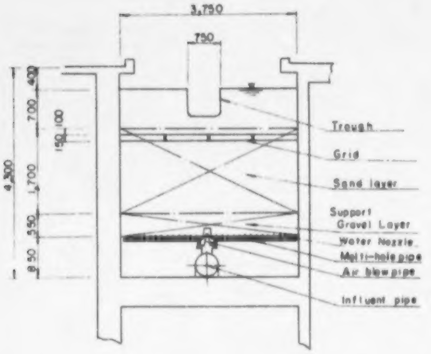


Fig. 3.1.15 Cross Section of Tri-Media Filter

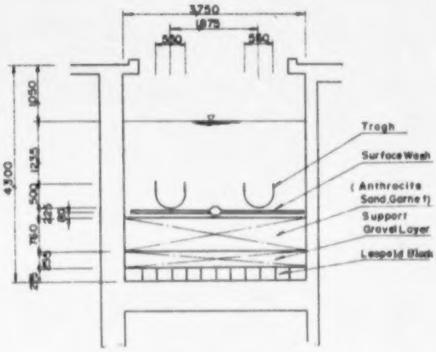


Table 3.1.7 Filter Media Characterization of the Upflow Filter

	Name	Specific Gravity	Depth (mm)	Particle Size Range (mm)	Effective size (mm)	Uniformity Coefficient
Filter Medium	Sand	2.63	1,700	0.8 ~ 2.0	1.2	1.27
Support media	Gravel	2.65	250	2.0 ~ 3.0	—	—
	Gravel	2.65	250	8.0 ~ 12.0	—	—
	Gravel	2.65	150	40.0 ~ 50.0	—	—
Total Depth			2,350			

Table 3.1.8 Filter Media Characterization of the Tri-Media Filter

	Name	Specific Gravity	Depth (mm)	Particle Size Range (mm)	Effective size (mm)	Uniformity Coefficient
Filter Media	Anthracite	1.39	420	0.84 ~ 2.0	1.22	1.41
	Silica Sand	2.62	230	0.42 ~ 0.84	0.42	1.46
	Garnet	4.05	110	0.18 ~ 0.42	0.29	1.53
Total Depth			760			
Support Media	Garnet	4.05	75	1.5 ~ 3.5	2.0	1.80
	Gravel	2.65	60	5.0 ~ 10.0	—	—
	Gravel	2.65	60	10.0 ~ 20.0	—	—
		2.65	60	20.0 ~ 30.0	—	—
Total Depth			1,015			

b. Experimental Results

(1) Water quality test results

From the middle of August through September of 1974 (summer season), the both filters were operated at various filter rates; 100 m/day (1.7 GPM/SF), 200 m/day (3.4 GPM/SF), 300 m/day (5.1 GPM/SF), 400 m/day (6.8 GPM/SF) and 500 m/day (8.5 GPM/SF) and a test at one filter rate was continued for 3 weeks. From November 14 to December 26 of 1974 (winter season), the filters were operated at the constant filter rate of 300 m/day (5.1 GPM/SF). Sampling time was 10:00 a.m. every morning. The average values obtained from the chemical analysis of samples of this test are listed in Table 3.1.9. Numbers of samples were 4 ~ 6 for each of a given filter rate in summer season and about 14 in winter season. In addition to the measurement of the above quality characteristics, hourly variations of BOD, COD and SS were observed as shown in Fig. 3.1.16 through 3.1.20. From these analyses, the following results were obtained:

Fig. 3.1.16 Water Qualities with Time
(Filter Rate 100 m/d)

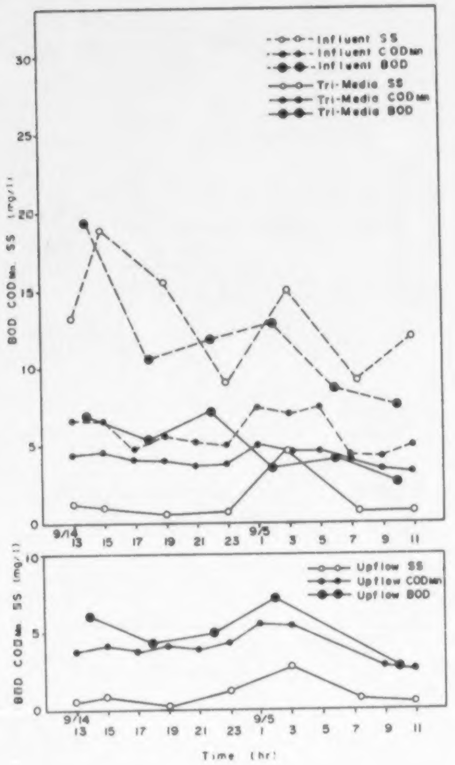


Fig. 3.1.17-1 Water Qualities with Time
(Filter Rate 200 m/d)

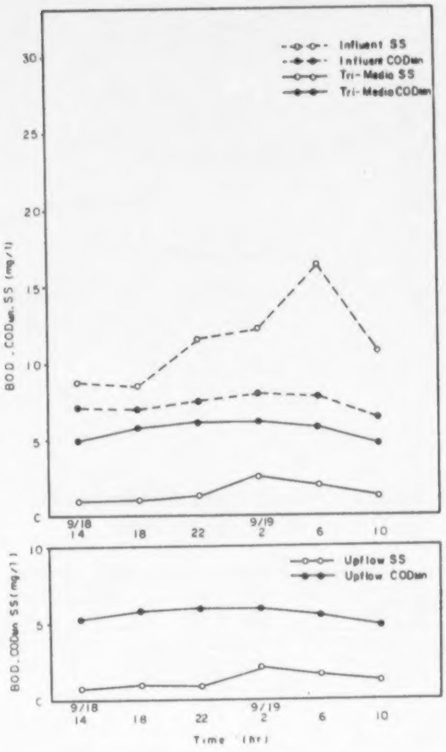


Fig. 3.1.17-2 Water Qualities with Time
(Filter Rate 200 m/d)

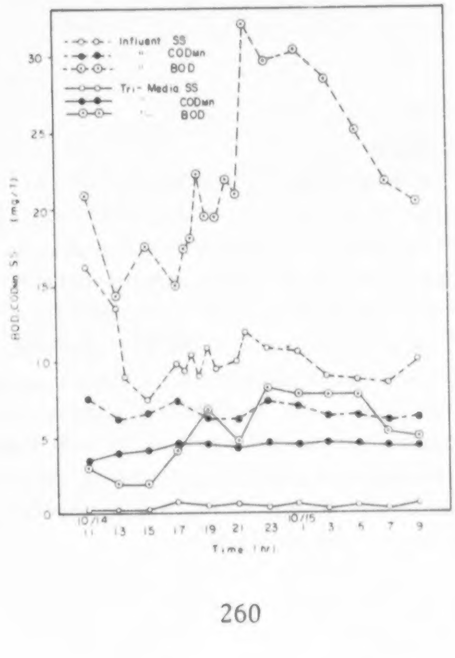


Fig. 3.1.18 Water Qualities with Time
(Filter Rate 300 m/d)

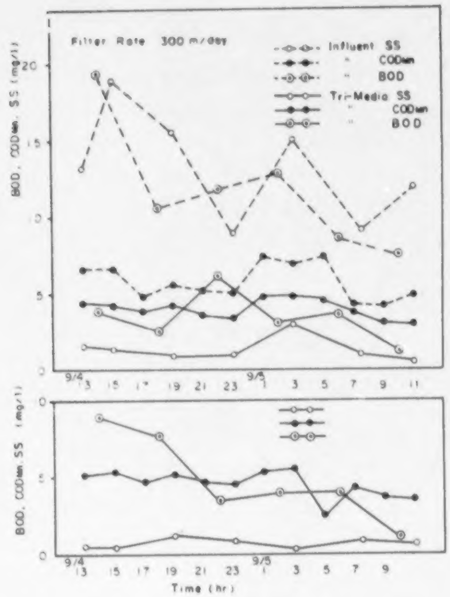


Fig. 3.1.19 Water Qualities with Time
(Filter Rate 400 m/d)

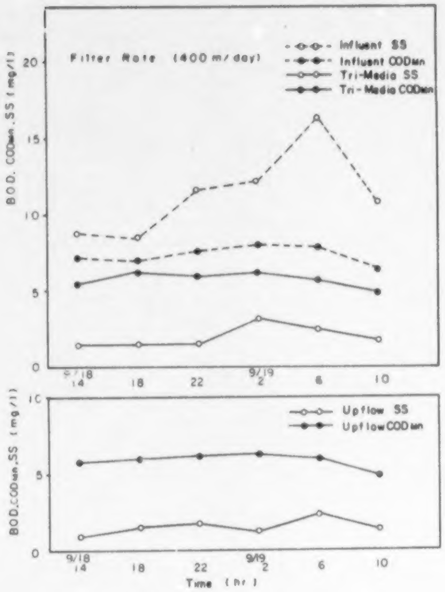


Fig. 3.1.20 Water Qualities with Time (500 m/d)

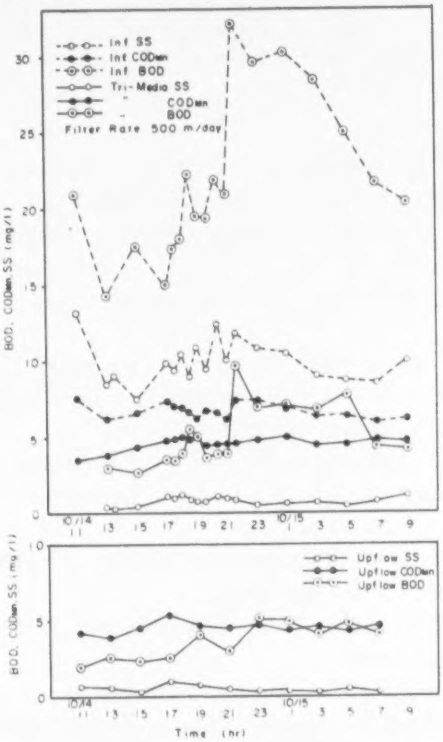


Table 3.1.9 Water Quality Characteristics

Filter rate		100 m/d		200 m/d		300 m/d		400 m/d		500 m/d	
		Upflow	Tri-Media	Upflow	Tri-Media	Upflow	Tri-Media	Upflow	Tri-Media	Upflow	Tri-Media
SS	Influent	9.1		6.9		9.1 (9.7)		6.9		7.3	
	Effluent	0.7	0.7	0.6	0.4	0.7 (0.9)	0.9 (1.2)	0.9	0.9	0.9	0.8
	Removal	92	92	91	94	92 (91)	90 (88)	87	87	88	87
COD Cr	Influent	21.1		21.9		21.1 (31.7)		21.9		23.5	
	Effluent	15.7	16.2	16.0	15.9	16.5 (19.5)	16.5 (20.9)	17.1	16.9	17.3	16.9
	Removal	26	23	27	27	22 (39)	22 (34)	22	23	26	28
COD Mn	Influent	5.6		5.0		5.6 (6.7)		5.0		5.7	
	Effluent	4.7	5.0	4.5	4.3	5.1 (5.4)	4.5 (5.7)	4.7	4.4	4.2	4.1
	Removal	16	11	10	14	9 (19)	20 (15)	6	12	26	28
BOD	Influent	8.6		7.1		8.6 (15.6)		7.1		16.9	
	Effluent	3.2	3.3	2.3	4.4	2.5 (3.9)	1.9 (5.5)	2.8	2.8	3.6	4.4
	Removal	63	62	68	38	71 (75)	78 (65)	61	61	79	74
T-N	Influent	8.2		7.9		8.2 (14.2)		7.9		8.6	
	Effluent	7.6	7.8	7.6	7.8	7.6 (13.6)	8.1 (13.3)	7.5	7.5	8.8	8.8
	Removal	7.3	4.9	3.8	1.3	7.3 (4.2)	1.2 (6.3)	5.1	5.1	7.4	7.4
K-N	Influent	5.6		5.3		5.6 (13.2)		5.3		7.0	
	Effluent	4.7	5.0	4.2	4.6	4.8 (12.1)	5.5 (11.8)	4.2	4.2	6.0	6.1
	Removal	16	11	21	13	14 (8)	5 (11)	21	21	14	13
NH3-N	Influent	3.6		3.6		3.6 (9.6)		3.6		4.5	
	Effluent	3.1	3.5	2.8	3.3	3.2 (9.1)	3.8 (8.5)	2.5	3.0	4.3	4.1
	Removal	14	3	22	8	11 (5)	-6 (12)	31	18	4.4	8.9
NO2-N	Influent	0.09		0.07		0.09 (0.05)		0.07		0.09	
	Effluent	0.48	0.52	0.63	0.36	0.54 (0.12)	0.14 (0.31)	0.62	0.53	0.28	0.08
	Increase	530	580	900	510	660 (240)	160 (620)	890	760	200	-10
NO3-N	Influent	2.91		2.60		2.91 (0.97)		2.60		2.39	
	Effluent	2.72	2.61	2.82	2.81	2.63 (1.45)	2.89 (1.14)	2.73	2.79	2.45	2.66
	Increase	-7	-10	8.5	8.1	-9.6 (60)	-0.7 (28)	5.0	6.6	2.5	11.3
DO	Influent	5.1		5.2		5.1 (6.1)		5.2		5.0	
	Effluent	2.6	0.9	1.3	0.4	2.3 (6.0)	3.6 (2.5)	0.8	0.6	1.3	2.0
	Decrease	49	83	75	92	55 (1.6)	29 (59)	85	89	74	60
ABS	Influent	0.16		0.23		0.16 (0.56)		0.23		0.37	
	Effluent	0.17	0.17	0.26	0.27	0.16 (0.19)	0.17 (0.38)	0.21	0.23	0.37	0.37
	Removal	-6.3	-6.3	-13	-7	0 (66)	-6 (32)	22	0	0	0
PH	Influent	7.2		7.1		7.2		7.1		7.2	
	Effluent	7.2	7.1	7.1	7.1	7.1	7.2	7.1	7.1	7.2	7.3
T-P	Influent					(4.73)					
	Effluent					(3.95)	(4.26)				
	Removal					(16.5)	(9.9)				

1) Unit for all data except the removal · is mg/l

2) removal is expressed by %

1) Regarding the effluent quality characteristics, there was no significant difference found between the effluents from the upflow filter and the tri-media filter.

2) No difference of the effluent qualities from the both filters was caused by the variation of the filter rate.

3) The both filters could produce the stable effluent qualities against the influent quality fluctuation.

4) SS concentration in the effluents from the both filters averaged less than 1.0 mg/ℓ during the summer season and less than 1.2 mg/ℓ during the winter season (average SS removal: 90% on the both systems).

5) The effluent BOD concentrations from the both filters averaged less than 4.4 mg/ℓ in the summer and less than 5.5 mg/ℓ in the winter (average BOD removal ratio: 60 ~ 80%).

6) Despite of the significant BOD reduction, COD_{Cr} and COD_{Mn} removals were as low as 20 ~ 25% and 10 ~ 20% respectively.

7) Nitrogen removals were 1 ~ 7% in Total-nitrogen, 10 ~ 20% in both Kjeldahl-nitrogen and Ammonia-nitrogen.

8) Nitrification took place in the filters in some degrees.

9) ABS removal could not be expected through the filter systems.

10) pH change was not observed through the sand filter system.

(2) Cleaning of filter basins

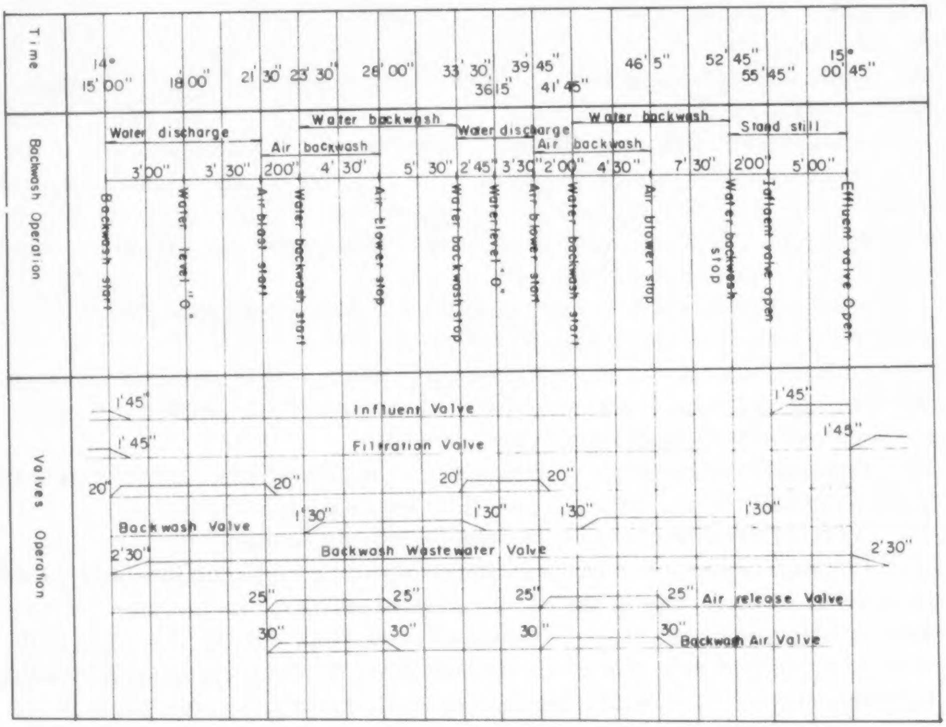
When filtrations are continuously conducted, suspended solids in the raw water are filtered and stored in the filter bed. As the amount of suspended solids stored in the filter bed increases, a filter head loss rises and finally reaches a terminal head loss because the pore spaces of the filter media are clogged with the suspended solids. In such filter condition, it is difficult to obtain the designed filtration rate. Then, it is required to perform an operation of washing the filter bed and cleaning the suspended solids stored in it. This operation is so called backwash.

The backwash program for both filters can be carried out manually or automatically. The automatic filter backwash is controlled either by time period of filtration, filter head loss and effluent turbidity. And also any patterns of backwash procedure can be selected with a pinboard timer that has 6 variations and 30 steps. A desirable backwash may be such that initial head loss does not increase. The employed backwash procedures are as follows.

1) Backwash procedure for the uppflow filter

An example of a backwash program is shown in Fig. 3.1.21. Here will be described the explanation of each stages.

Fig. 3.1.21 Time Schedule of the Upflow Filter Backwash



i) Backwash start: Filtration run is terminated on receiving the backwash start signal and the pinboard timer start.

ii) Water drainage: When the backwash stage starts, the filter influent and effluent valves close and the drainage valve and the backwash waste water valve open so that the water level in the basin lowers.

iii) Air blow start: After water level is read to be zero by the level meter (air purge system) positioned at 10 cm below the filter surface, another timer works and in 4 min. the air blower starts. During the air blow, the pinboard timer stops. Water in the bed is completely withdrawn within 4 min.

iv) Water backwash start: in 2 min. after the air blow starts, a backwash water pump starts. Because it takes some time until the air blower achieves the determined flow. Backwash flow is as low as 10 m^3/m^2/hr. in first 5 min., after then it increases up to 40 m^3/m^2/hr. The time when the backwash flow changes from 10 to 40 m^3/m^2/hr. is synchronized with the end of the air blow. The backwash in the low flow has the purpose of extending the time of air-water phase as long as possible and of discharging suspended solids as much as possible.

v) Air blower stop: When backwash water level rises up to the point of 20 cm below the top of the trough, the air blow stops. This is so as to prevent the sand media flowing out. The working time of the blower is generally 7 min.

vi) Water backwash stop: The backwash pump stops having worked for 10 min. after its start.

vii) Water drainage: The drainage valve opens simultaneously when the backwash pump stops. Then, all water in the bed is discharged with the same operation as the step of ii).

viii) Air blow start.

ix) Water backwash start: Water is sent in the same procedure as iv), from the low flow to the general flow.

x) Air blow stop: The same operation as iv), the operation time is almost 7 min.

xi) Water backwash stop: The operation time for the second water backwash is 2 min. longer than that for the first water backwash. It is continued for 12 min. The longer time is taken with the purpose of completely discharging suspended solids and air remained in the filter bed.

xii) Standstill time: The backwash operation is kept still for 2 min. after the backwash pump has stopped in order to settle the filter media.

xiii) Filtrate discharge: An influent valve opens and a next filtration service starts. But the backwash wastewater valve is kept open during 5 min. and the first portion of the filtered water is discharged from the trough.

xiv) Termination of backwash process: The effluent valve opens and the backwash waste water valve closes. Then, the next filter service starts.

2) Consideration of the backwash system for the upflow filter

i) The main consideration of the backwash procedure for the upflow filter should be given to the final stage of the backwash; stand still time. In this stage, the backwash operation is paused after the backwash pump stops and the water level in the filter is maintained as it is in order to settle down the sands expanded during the backwash process. In view of dense packing of filter media, this stage has an important relation with a breakthrough of suspended solid from the bed which is discussed in the section (8). Then, the perfect compaction is intended by opening the drainage valve and discharging the remained water. Moreover, to keep the drainage valve open is effective to absorb the excessive high pressure caused temporarily when the influent valve opens at the rebeginning of filtration and to prevent the rapid change of the pressure imposed to the sand media for the purpose of its stabilization. A time requirement to decrease the raw water contained in the headchamber (maximum waterhead; 3.0 m) to the constant water level is the reason why the high pressure is produced.

Since August 6, 1975, the following operation was added to the final stage of the operations shown in Fig. 3.1.21. In 2 min. after the pause, the drainage valve is opened. When the water in the filter basin is discharged to just below the sand surface, the influent valve is opened and the raw water of the amount for filtration is sent in. After 5 min. of discharging the filtered water out of the backwash wastewater valve, the effluent valve opens and the filtration starts again.

ii) Increase of backwash water amount: General flow is increased to 1,400 m^3/h ($0.78\ m^3/min.m^2$) for treatment of the raw water with high turbidity.

3) Backwash procedure for the tri-media filter

Fig. 3.1.22 shows an example of backwash for tri-media filter. Here will be described the explanation of each stages.

Fig. 3.1.22 Time Schedule of the Tri-Media Filter Backwash

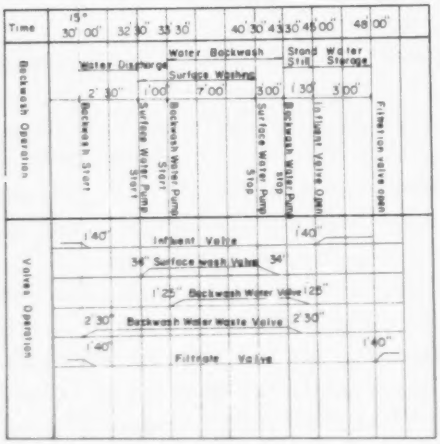
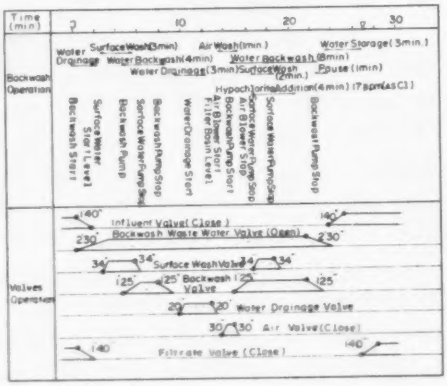


Fig. 3.1.23 Time Schedule of the Tri-Media Filter Backwash (for High Turbidity)



i) Backwash start: On receiving the backwash start signal, the pinboard timer starts.

ii) Water drainage: The effluent and influent valves close and the backwash waste water valve opens. Thereby, water in the basin is discharged.

iii) Surface wash start: The water level lowered to the top end of the trough, the level meter detects the water level and the signal is sent to start the surface wash pump. During this time the pinboard timer stops.

iv) Water backwash start: 1 min. after the surface wash pump starts, the water backwash pump starts.

v) End of surface wash: The surface wash pump stops after the 8 minutes' operation. The surface washing and water back-washing works are simultaneously conducted for 7 min. This simultaneous operation is essential to prevent suspended solids removed on the bed surface from producing mudball. Deficiency of surface washing may cause mudball forming because the suspended solids remained on the dead area of the bed surface agglomerate anthracite particles.

vi) Water backwash end: The water backwash pump stops working in 10 min. after its start; it continues working for 3 min. more after the surface wash pump stopped. This is for the purpose of discharging completely all the solids in the filter bed.

vii) Operation pause: The filter is kept still for a minute after the end of water backwash in order to give enough time for the filter bed settling down.

viii) Water storage: The influent valve opens and the backwash wastewater valve closes. Then water is stored until water level rises up to the level for a normal filter service.

ix) Termination of backwash: The filter backwash is terminated by opening the effluent valve.

4) Consideration of the backwash system for the tri-media filter

A major problem in the backwash system for the tri-media filter exist in the slime production in the filter material, especially in the sand media. This tack slime production may cause the low fluidity of the filter media in the backwash step so that an effective cleaning of the bed can not be achieved. In order to solve this problem, the backwash procedure was changed as follows.

i) Sodium hypochlorite is added during the water backwash.

When cleaning is repeated without an addition of hypochlorite, the slime is produced in the sand layer and the fluidity of the sand is lowered. On this account, the initial head loss gradually increases. The initial head loss increased about 10 cm (Filtration speed: 200 m/day) for 2 months from June 4 to August 8, 1975 as shown in Fig. 3.1.25. This period is the season when the temperature becomes gradually higher. Therefore, when the viscosity of water is lowered and the initial head loss generally tends to decrease, we may conclude that this increase of the initial head loss was not influenced by the temperature. The addition of sodium hypochlorite started from August 8, 1975. Its addition starts 2 min. after the full opening of the backwash valves and stops in 4 min. All the above operations are automatically conducted. The adding point is just before the backwash pipe enters the filter basin. The added amount is 250 ℓ/h (1.1 GPM). A percentage of effective chlorine in the sodium hypochlorite solution is 10% so that the chlorine injected concentration is 17 mg/ℓ. As the effective capacity of the storage tank of sodium hypochlorite (made from F.R.P. added polyethylene, with the cylindrical cap) is 2,000ℓ (528 G), the supplement of the sodium hypochlorite is required once 2 months for the operation of two filter basins (a basin; 30 m^3). After the cleaning with an addition of sodium hypochlorite continued for a month since August 8, 1975, the initial head loss was lowered by 30 cm. This has proved the effectiveness of the above described cleaning method.

ii) Air-water phase cleaning

Mixing cleaning of air and water is additionally performed after water is drained out of the basin.

On August 16, 1975, the filtration of the raw water with high turbidity has begun (in mixing with the drained secondary sedimented sludge). After about a month, the condition of the filter layer of one of the two basins has become worse. SS accumulated on the anthracite surface layer was not cleaned off, but fell down along the wall surface, and reached the supporting gravel layer. It was found that the cleaning system shown in Fig. 3.1.22 has been insufficient for this kind of raw water with high turbidity. Then, air-water mixing cleaning was conducted and also the quantity of backwash water was increased up to 1,500 m^3/h (0.83 $m^3/min.m^2$) (20 GPM/SF). The quantity of air used for this purpose was to be 15 $Nm^3/min.$ (0.5 $Nm^3/min.m^2$) (1.64 GPM/SF). In this way the raw water with middle strength of turbidity was filtered from October 21 to November 3, 1975, but the above written deterioration of the filter performance was not observed. The altered backwash system is described in Fig. 3.1.23.

i) Backwash start and drainage: On receiving the backwash start signal, the filtration process is ended and the backwash process starts. In the first step of the pinboard timer, the influent and effluent valves close and the backwash wastewater valve opens. This state is kept until the water level is lowered to the design level that the surface washing can be performed.

ii) Surface wash and water backwash: The first cleaning is with the purpose of flowing out SS accumulated on the surface of the filter bed and consists of 3 minutes' surface washing and four minutes' water backwashing (overlap of 1 min.).

iii) Drainage: The drainage valve being opened, water level of the filter basin is lowered charged to the bottom out of the filter basin, in order to perform the following air-water mixing backwash effectively.

iv) Air-water mixing backwash: The air blower starts first, the backwash pump starts next, and the air-water mixing backwash is performed with pushing up the backwash water to bed surface. The air blower is driven only for a minute. Then the water surface is just above the anthracite so that there is no fear that the filter media are carried over the trough. The purpose of this air-water mixing backwash is to clean out SS within the filter bed.

v) Surface layer wash: This is conducted by turning the surface washing equipment during the operation of the water backwash pump. The surface washing equipment is to be turned within the expanded layer of anthracite so that the water backwash, and the cleaning of the inner portion of the anthracite layer is performed effectively.

vi) Chlorine addition: The addition of sodium hypochlorite starts 2 min. after the second backwash pump start and it continues for 4 min. The hypochlorite addition is not sufficiently effective in the first backwash because the large part of hypochlorite is consumed by SS remained in the filter bed.

vii) Pause and water storage: After settling down the filter media, the influent valve opens and the backwash wastewater valve closes. Then water is stored and after the water level is raised to the level for a regular filtration, the effluent valve opens and the filtration starts again.

5) Quantity of backwash water

The design flow and the actually used flow for the backwash of each filters are presented in Table 3.1.10.

Table 3.1.10 Quantity of Backwash Water

		Design Flow		Total Amount
		per basin	per basin	
the Upflow Filter	Backwash Water (Raw Water)	300 m^3/H 1,200 "	10 m^3/H 40 "	347 m^3
	Backwash air	45 N - m^3/H	1.5 N - m^3/H	440 N - m^3
the Tri-Media Filter	Surface wash Water (Filtrate)	45 m^3/H	1.5 m^3/H	61 m^3
	Backwash Water (Filtrate)	1,100 m^3/H	36.7 m^3/H	172 m^3

(3) Initial head loss variation

A head loss produced at the rebeginning of filtration run after the termination of the filter backwash is so called the initial head loss. The continuous investigation of the initial head loss testifies a degree of the filter bed cleaning. If an ideal backwash could be achieved, an increase of the initial head loss may not be observed. Adversely an increase of the initial head loss may indicate an incomplete cleaning of the filter bed. The initial head loss variation of the upflow and the tri-media sand filters from August 15, 1974 to March 15, 1976 was investigated.

1) Initial head loss of the upflow filter (Fig. 3.1.24)

A stable condition of the initial head loss was observed until August 15, 1974. It might be associated with a chlorine addition of 3 mg/ℓ to the influent. After then, the chlorination of the influent was stopped to obtain a stable effluent quality and a filter rate was increased to 400 m/day (6.8 GPM/SF), 500 m/day (8.5 GPM/SF), and 550 m/day (9.4 GPM/SF).

A rapid increase of the initial head loss was experienced in this test period. So, chlorine solution was temporarily added to the influent on November 6. The initial head loss was maintained to be low for several days since chlorine was added. But after that the initial head loss began increasing again rapidly. In the previous wash system, the air blow had been carried out in the condition that water was filled in the filter bed. In the modified wash way, however, water is discharged initially down to the bottom of the bed and the backwash is conducted from the low water flow to the regular water flow as described in the section of (2) - 1). In this way, the initial head loss was maintained to be low without the chlorine addition as shown in Fig. 3.1.24.

From January 7, 1975, the period of the backwash time was shorten to a half of the usual one, but the low initial head loss was not maintained. Also, it was found that a constant initial head los could stably obtained by providing the backwash once a day. Moreover, the filter was operated at the standard filter rate (200 m/day) for 4 months from the begining of June to the middle of October of 1975. In this period the initial head loss was constantly maintained at 30 cm. This proves that the above suggested backwash method can be applied for the practical operation.

Fig. 3.1.24 Initial Head Loss Variation of the Upflow Filter

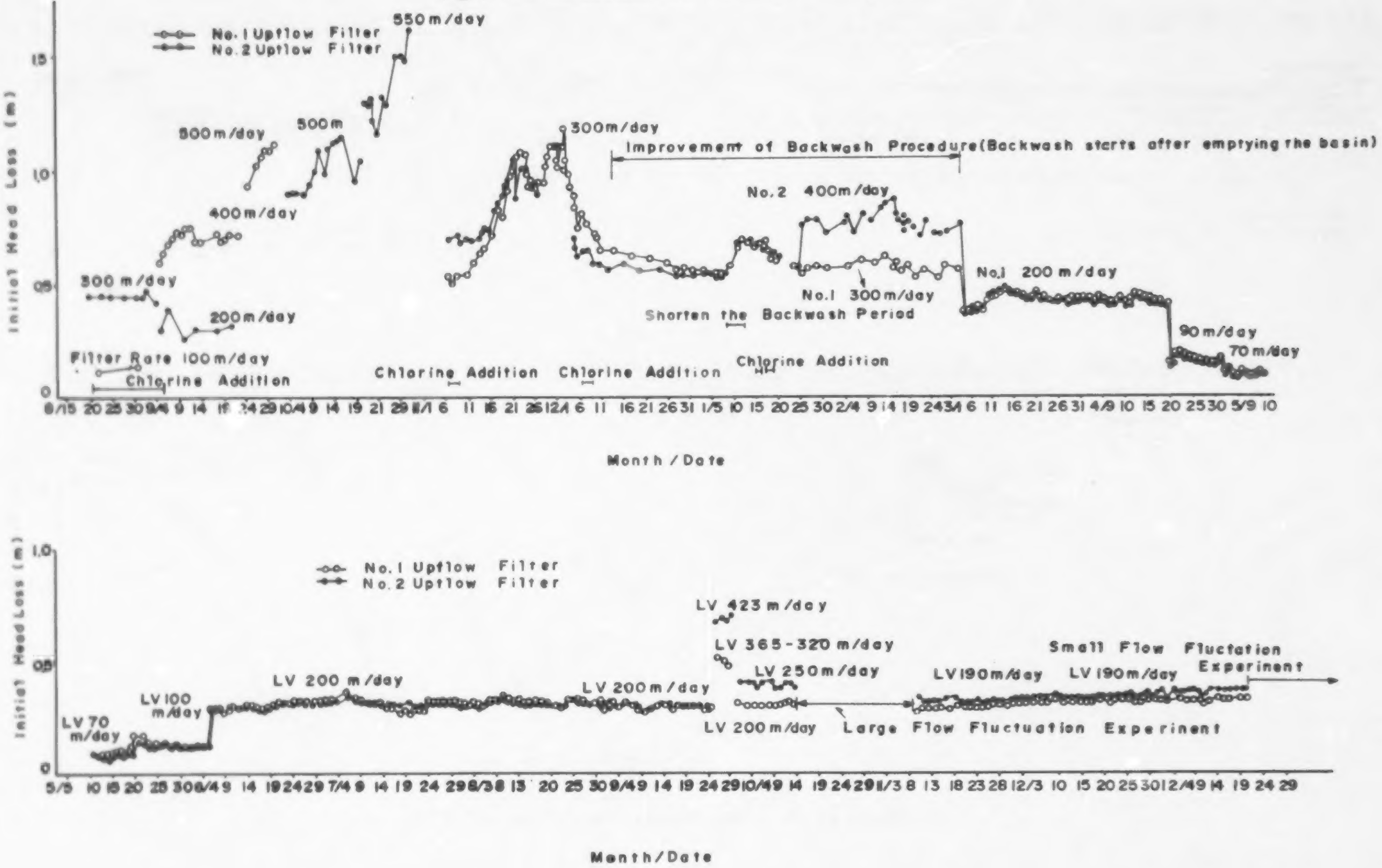
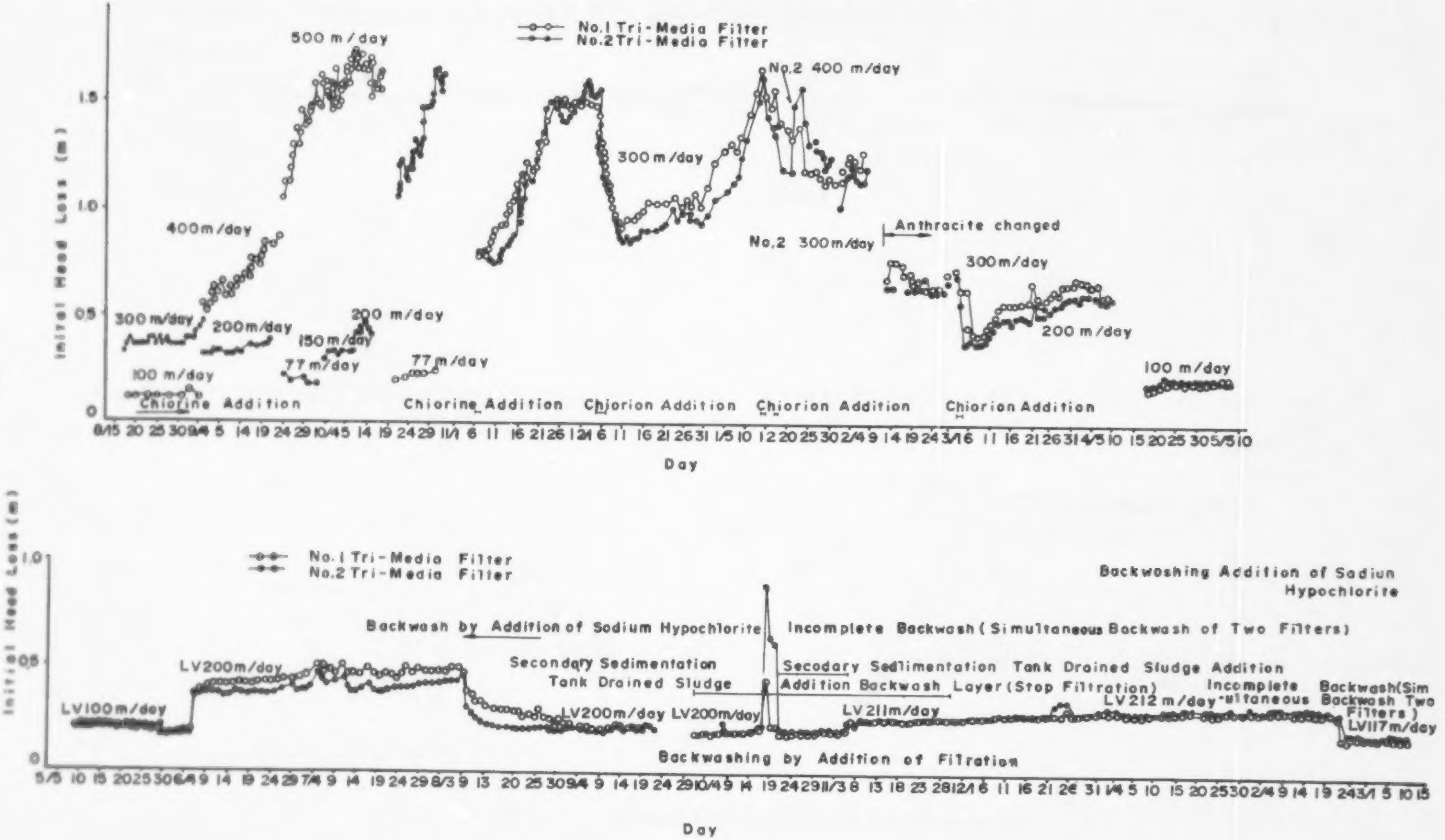


Fig. 3.1.25 Initial Head Loss Variation of the Tri-Media Filter



2) Initial head loss for the tri-media filter (Fig. 3.1.25)

In the operation of the tri-media filter, no addition of chlorine showed an increase of the initial head loss as well as in the upflow filter operation. A change of the backwash system, in this case, was not effective to prevent the initial head loss increase. When the operation was performed with the backwash of once a day at the standard filter rate (200 m/day) (3.4 GPM/SF), the initial head loss increased about 10 cm by August 8 (in about two months). Then, in the backwash system, sodium hypochlorite was injected for 5 min. in 4 min. after the backwash start. The injection rate of effective chlorine was 10 ppm. The initial head loss was lowered about 30 cm on August 9 in a month after this backwash start and since then the initial head loss could be maintained at about 20 cm. Thus, the backwash system with the addition of sodium hypochlorite made the stable practical operation possible for tri-media filter.

(4) Filtration period and initial head loss

Concerning the hourly variation of the filter head loss and the effluent turbidity, the data obtained at a filter rate of 300 m³/day (3.4 GPM/SF) are shown in Fig. 3.1.26, which indicates that the effluent turbidities from the both filters were not influenced with the fluctuation of influent turbidity. However, there was a great difference between the upflow and the tri-media filter with respect to the time period of filtration run that could be applied. Fig. 3.1.27 shows the relationship between a terminal head loss and the filtration period at various flow rates. The terminal head loss means the head loss reached at the end of the filtration run.

Fig. 3.1.26 Head Loss Variation and Turbidity Characteristics in the Upflow and the Tri-Media Filters

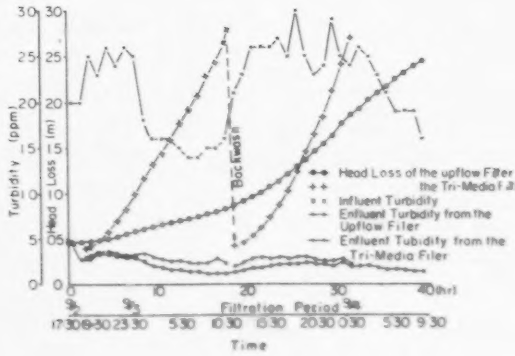


Fig. 3.1.27 Relationships of the Filtration Period and the Terminal Head Loss on the Upflow Filter Operation

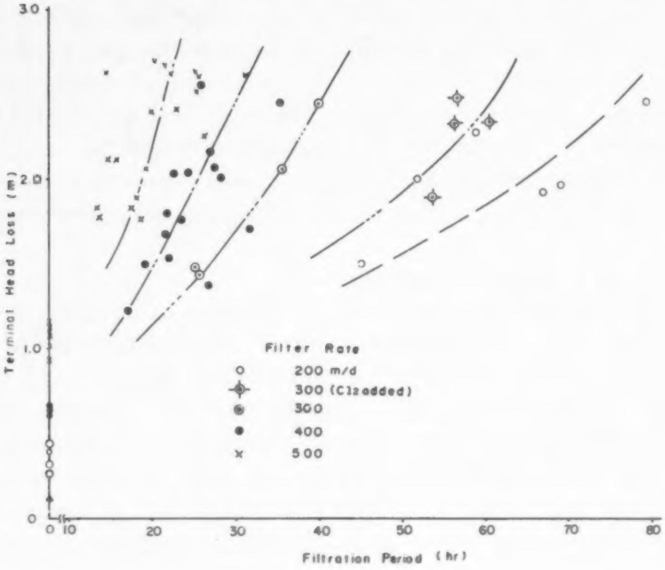


Fig. 3.1.28 Relationships of the Influent Turbidity and the Filtration Period on the Tri-Media Filter

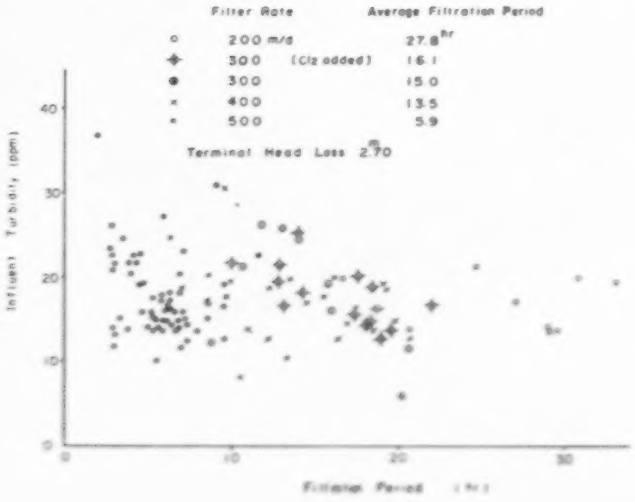
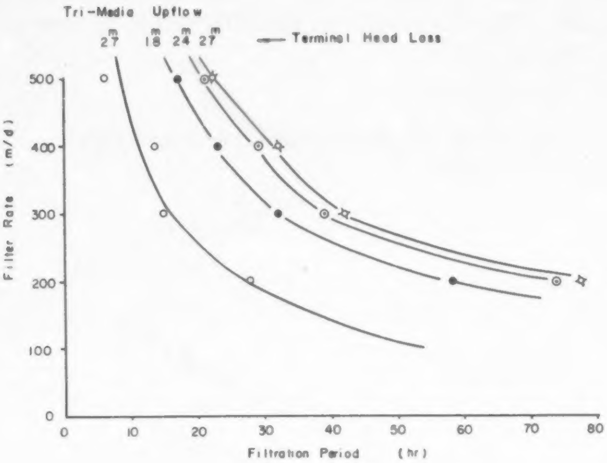


Fig. 3.1.29 Filtration Periods of the Upflow and Tri-Media Filter



At the beginning of the experiment, there was a difficulty to employ a high terminal head loss because of some structural problems and also a breakthrough problem. Subsequently, an instable terminal head loss was observed. The structural problems were of the physical damages of the FRP trough. They have been reconstructed and now working well. The filtration periods applied to the tri-media filter are presented in Fig. 3.1.28. Also, the average filtration periods at various filter rates are presented in Fig. 3.1.29, which indicates that at every rates studied the upflow filter has a longer filtration period than the tri-media filter has.

(5) Suspended solids removal capacity

The removal capacity of suspended solids in the experiment performed from August 15 to October 30 of 1974 (in this period, a filter rate was varied in a range from at 100 m/day (1.7 GPM/SF) to at 500 m/day (8.5 GPM/SF) was calculated by using the following equation.

$$M = Q\,(T_r \times r_r - T_f \times r_f) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Where,

M = Amount of SS removed in once filtration run (g)
T_r = Average turbidity of influent (mg/ℓ)
T_f = Average turbidity of effluent (mg/ℓ)
r_r = A correlation coefficient of turbidity and SS in the influent (0.50)
r_f = A correlation coefficient of turbidity and SS in the effluent
Q = A quantity of the filtered water during one filtration run (m^3)

The average turbidities (T_r, T_f) were obtained by measuring the influent and effluent turbidity at every 30 min. during the filter runs. The correlation coefficients of turbidity and SS were obtained from Fig. 3.1.30.

The amounts of SS removed by the upflow filter are presented in Fig. 3.1.31

and that removed by the tri-media filter in Fig. 3.1.32. The figures indicate that 150 kg (1.0 lb/SF) of SS could be removed by one upflow filter bed (30 m²) and 50 kg of it by one tri-media filter bed (30 m²) under the following operational conditions, filter rate; 300 m/day, influent turbidity; 20 ppm, terminal head loss of the upflow filter; 2.40 m.

Fig. 3.1.30 Correlation between Turbidity and SS

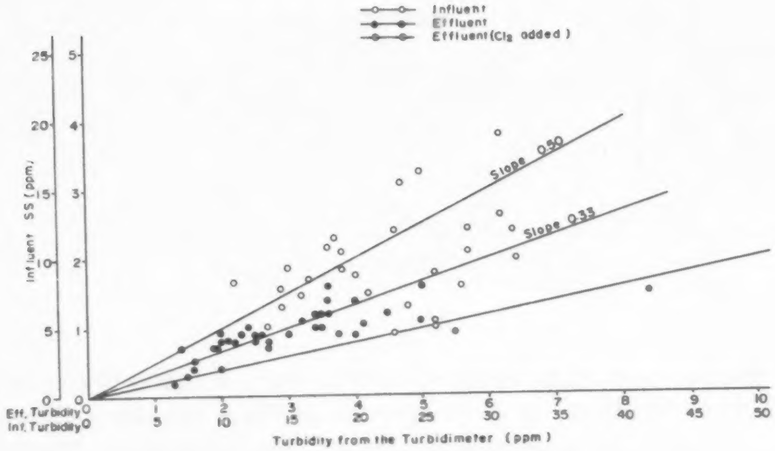


Fig. 3.1.31 Terminal Head Loss and Removed Suspended Solids by the Upflow Filter

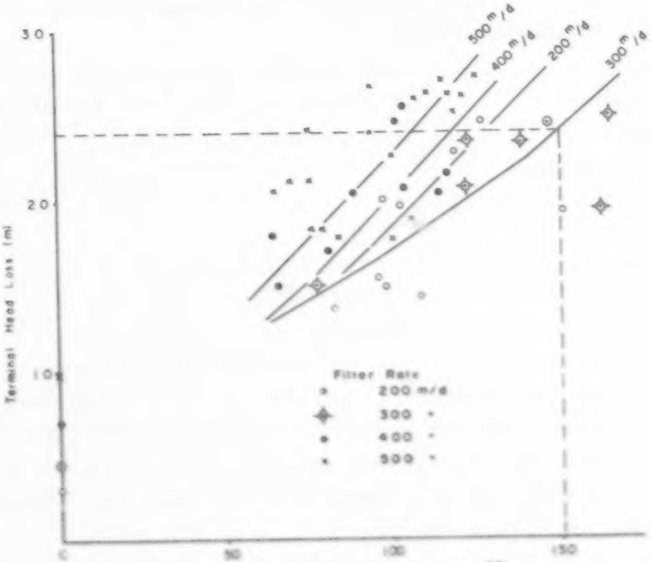


Fig. 3.1.32 Influent Turbidity and Removed Suspended Solids by the Tri-Media Filter

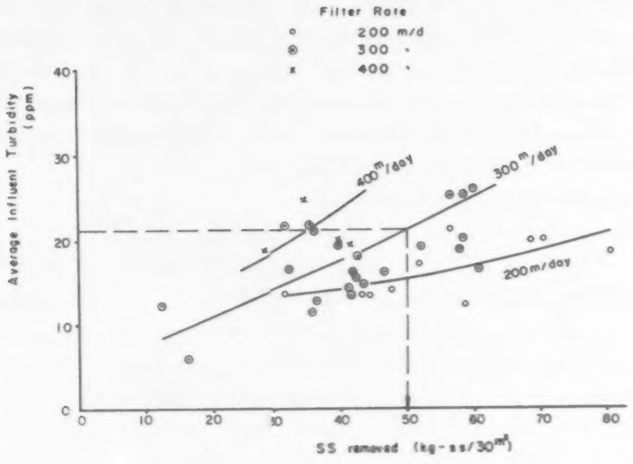
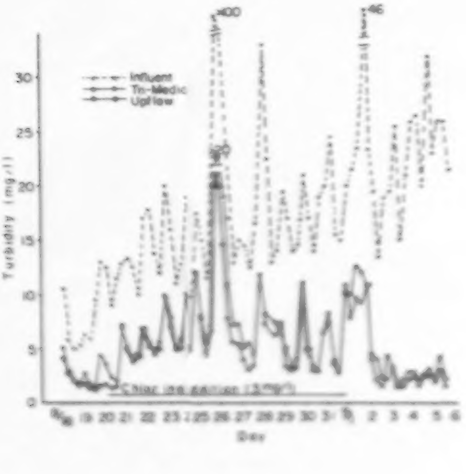


Fig. 3.1.33 Influent and Effluent Turbidities at Filter Rate of 300 m/d



(6) Chlorine addition and effluent turbidity

For the control of slime generation in the sand filter, chlorine is generally added to influent. However, the chlorine addition resulted in unstable effluent quality. Then, chlorine addition was, principally not conducted and also other means such as modifying the backwash system for the upflow filter and adding sodium hypochlorite to the backwash water for the tri-media filter, were employed for the slime control in this experiment. Fig. 3.1.33 shows the turbidities variation in the influent and effluent from the both filters. Since the chlorine addition to the influents were carried out for the period from August 20 to August 31 of 1974, the effluent turbidities of the both filters showed the fluctuation. This indicates that it is largely influenced by the influent turbidity variation.

(7) Head loss distribution in filter media

Cross-section head loss distribution through the upflow filter bed are grafically shown in Fig. 3.1.34 and 3.1.35. Also, Fig. 3.1.36 and 3.1.37 show the head loss distributions of the tri-media filter. Fig. 3.1.34 shows that the graval layer at the bottom of the bed can be an effective filter medium in the upflow filter. It is also shown by Fig. 3.1.35 that the suspended solids once caught by the supporting graval layer gradually rise to the upper sand medium. The effective filter depth of the sand medium above the graval layer seemed to be around 30 cm.

Fig. 3.1.36 and 3.1.37 show that in the case of the tri-media filter, about 80% of the total SS removed was stored in the top layer of anthracite that was about 30 cm in depth. Both in the sand medium in the upflow filter and the anthracite medium in the tri-media filter, the effective filter depth was about 30 cm. However, in the upflow filter, the graval layer at the bottom of the bed had an effective filter function. So that, totally, the upflow filter is able to provide a larger SS storage capacity than the tri-media filter.

Fig. 3.1.34 Head Loss Distributions of the Upflow Filter (300 m/d)

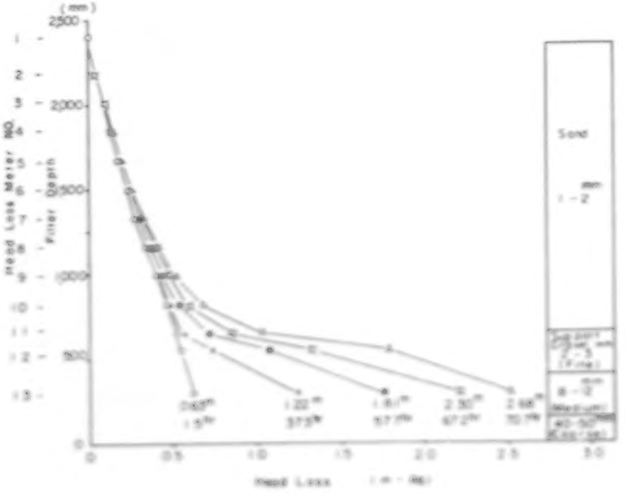


Fig. 3.1.35 Head Loss Distribution of the Upflow Filter (400 m/d)

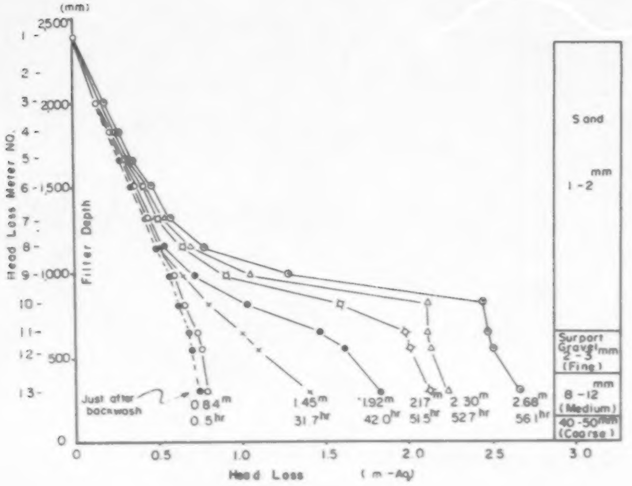


Fig. 3.1.36 Head Loss Distribution of the Tri-Media Filter (200 m/d)

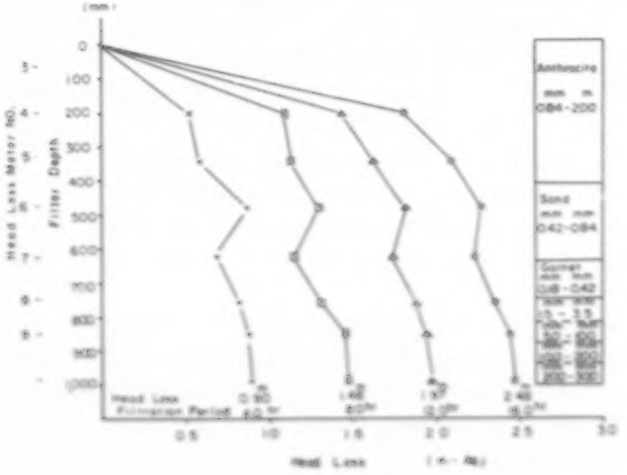
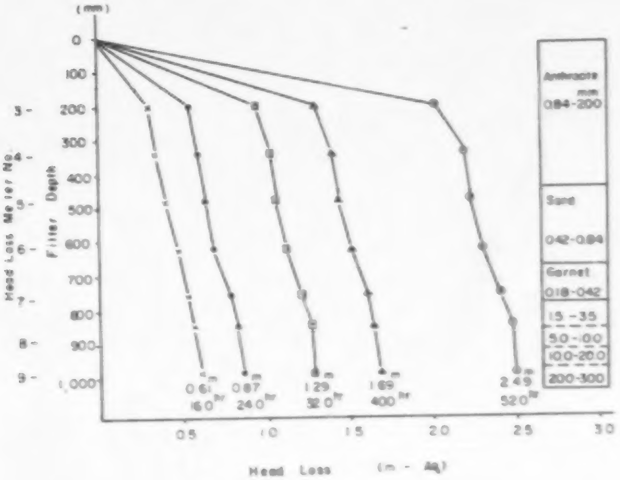


Fig. 3.1.37 Head Loss Distribution of the Tri-Media Filter (300 m/d)



(8) Breakthrough problem on the upflow filter operation

Filter breakthrough may refer to the phenomena that the stratified filter bed is partially broken and a water channel is temporarily made so that the stored SS at the bottom layer is released out of the bed during a filtration run. When the filter breakthrough occurred, a heavy turbid water can be observed above the surface of the filter bed and the filter head loss lowers rapidly. After a while, the breakthrough will spontaneously disappear and the head loss will increase again. However, when the head loss increases to the point that the last breakthrough happened, another breakthrough may occur again. Then, if the breakthrough once happened, the filtration run must be terminated to start the backwash operation.

The initial head loss and head loss at the breakthroughs occurred since the operation start till the present time are investigated and presented in Fig. 3.1.38, in which the term of normal head loss means the initial head loss shown while the filter rates were at 300 m/day (5.1 GPM/SF) and 400 m/day (6.8 GPM/SF) and the successful backwashing of the filter bed was carried out from January 25 to February 24 of 1975. Fig. 3.1.38 indicates that the following two may be considered as the reasons why the breakthrough phenomena happen.

1) A sufficient decrease of the initial head loss may not be achieved due to the deficiency of the filter backwash (in the case of 300 m/day).

2) Although the initial head loss is low enough, the standstill step at the end of the backwash operation may not be provided (in the case of 300 m/day and 400 m/day).

A stress generation in the grids was measured by mounting a strain gauge on the grid. The head loss observed at the beginning of the stress generation was ranged from 1.70 m to 1.76 m (from 5.6 to 5.8 FT), as presented in Table 3.1.11.

Fig. 3.1.38 also indicates that the head loss at the time of breakthrough occurring was above 1.80 m (5.9 FT).

A mechanism of the breakthrough phenomena may be explained as follows, based on the above investigations.

As the amount of stored SS increases and the clogging of the filter media proceeds, the sand layer may receive the upward force which is larger than the weight of the sand bed itself. Then, when the sand particles in the top sand layer are uniformly arranged, the inverse bridge is formed with the grid and the force to push the sand bed up may be transmitted from the sand medium to the grids. Thus the sand layer can support this force. But when the backwash is insufficient and the sand particles are coated with slime, or when the compaction of the sand layer by the pause stage is insufficiently performed, the resistive force due to the engagement of the sand particles can not be expected and the upward force might be released through the portion of the sand particles. Eventually, the breakthrough of the removed suspended solids may take place.

Fig. 3.1.38 Head Loss at the Time of Breakthrough Occurrence and Initial Head Loss

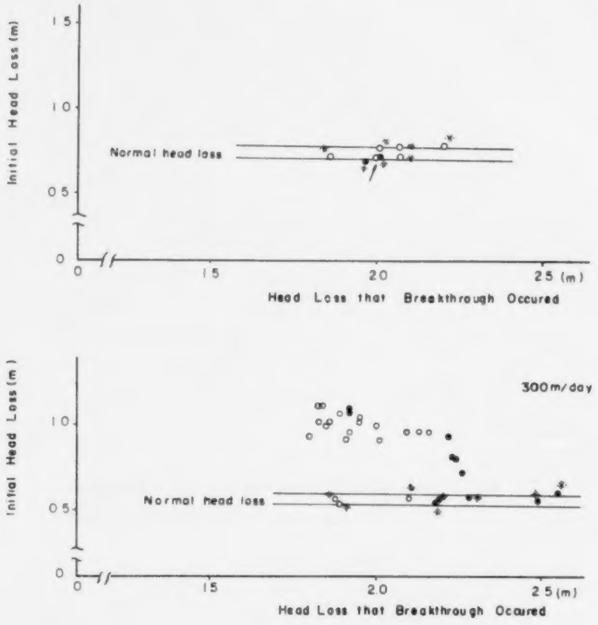


Table 3.1.11 Head Loss when Stress Generated in Grid

Measure Date 1974	12-10	12-13	12-20	12-24	12-27	12-29
Head Loss (m)	1.71	1.73	1.76	1.71	1.73	1.70

(9) Consideration of the filter media used in the tri-media filter

Generally, suspended solids of influent are removed at the top layer of the tri-media filter, as described in the section of (7). Then in order to extend a filtration period, an effective depth of the filter must be increased. One of factors to extend a filtration period may exist in the particle size of anthracite in the top layer of the filter media.

Conventionally the anthracite with an effective particle size of 1.22 mm (uniformity coefficient 1.41) was employed. The anthracite with an effective size of 1.70 mm (uniformity coefficient 1.21) was to be employed instead of the above. Column test was carried out by using two columns filled with the anthracite of two particle sizes (The other media are equal to the existing ones). Test results are shown in Fig. 3.1.39. It indicates that the larger particle could increase the filtration duration as 1.7 times longer as the smaller one.

Fig. 3.1.39 Effects of Particle Size of Anthracite for the Tri-Media Filter

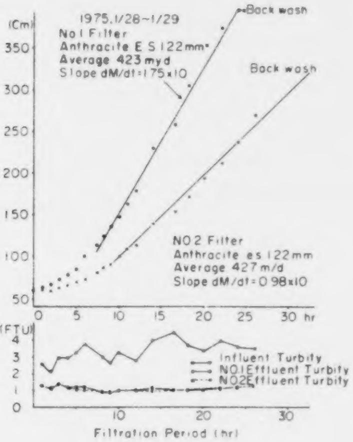
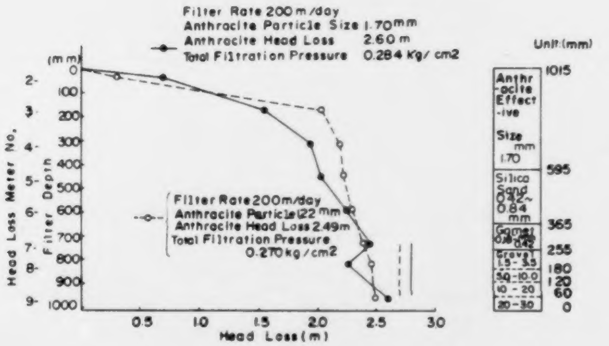


Fig. 3.1.40 Head Loss Distribution After the Change of Anthracite Particle Size



According to the above tests results, about 76% of the anthracite medium of the tri-media filter (32 cm of 42 cm anthracite medium depth) was exchanged with the larger anthracite with the effective size 1.70 mm. Fig. 3.1.40 shows that the anthracite with the effective particle size of 1.70 mm admits the deeper SS invation and the filter media are found to be more effective.

(10) Practical filter rate

To determine the operational filter rate, the decrease of an amount of the effluent caused by the backwash is not taken into consideration. Then, the following equation was used for obtaining a practical filter rate, which is termed as the filter rate that the backwash time requirement and the amount of the backwash water are taken into consideration.

$$Q = 24 \frac{F - W}{T_f + T_w} \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (2)$$

where,

Q = Practical filter rate (m/day)
F = Filter flow rate per unit surface area (m^3/m^2)
W = Backwash flow rate per unit surface area of the tri-media filter (It is not required for the upflow filter.) (m^3/m^2)
T_f = Filtration period (hrs.)
T_w = Backwash period (hrs.)

The filter flow rate, F is expressed by using the operational filter rate, Q′ (m/day), as follows;

$$F = \frac{1}{24} Q' \times T_f \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (3)$$

From the equation (2) and (3), the relationship of the practical filter rate and the operational filter rate is expressed by the following equation.

$$Q = \frac{Q' T_f - 24W}{T_f + T_w} \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (4)$$

Calculation results by using the above equation are presented in Table 3.1.12. It was proved that the upflow filter was superior to the tri-media filter both in the practical filter rate and in the operational filter rate.

Table 3.1.12 Practical Filter Rate

1975, 4/1 ~ 4/22

Operational Filter Rate	Practical Filter Rate (m/d)		
	The Upflow Filter		The Tri-Media Filter
m/day	ΔH : 1 80m	ΔH : 2 70m	ΔH : 2 70m
200	197 (15)	198 (1.0)	193 (35)
300	293 (23)	295 (17)	285 (50)
400	387 (32)	391 (23)	378 (55)
500	478 (44)	482 (36)	464 (72)

All numbers averaged 40 data of a filter rate

Number in a parenthesis means decrease %

(11) E. coli behavior in the filtration process

Usually, about 75% E. coli removal could be achieved by the both filter systems as shown in Table 3.1.13. Furthermore, it was very rare case that E. coli number in the effluents exceeded 600. Then, when the sand filter is employed for the treatment of the secondary effluent, the chlorination of the filter influent may be unnecessary.

Table 3.1.13 E coli Removal Characteristics

unit number /ml

Date	Influent		the Upflow Filter				the Tri-Media Filter			
			1		2		1		2	
	Caliform no	E Coli no	Caliform no	E Coli no	Caliform no	E Coli no	Caliform no	E Coli no	Califrom no	E Coli no
5/21						113				35
10 00		660				(83)				(95)
				146				56		
13 30		520		(72)				(89)		
5/27				330				350		
11 00		500		(34)				(30)		
								490		
13 00		1300		450				(62)		
						350		390		
15 00		900		(65)		(56)		(51)		
6/13			26×10^3	620	25×10^3	600	29×10^3	610	19×10^3	590
9 00	11×10^4	2 400	(76)	(74)	(77)	(75)	(74)	(75)	(83)	(75)
			19×10^3	500	26×10^3	510	24×10^3	500	16×10^3	480
11 00	5×10^4	2500	(62)	(80)	(48)	(80)	(59)	(80)	(68)	(81)
			Filter	Backwash						
11 05			37×10^3	520	38×10^3	580	42×10^3	550	28×10^3	520
14 00	15×10^4	2 300	(75)	(77)	(75)	(75)	(72)	(76)	(81)	(77)
			28×10^3	480	29×10^3	440	33×10^3	540	30×10^3	500
16 00	11×10^4	1 900	(75)	(75)	(74)	(77)	(70)	(72)	(73)	(74)

Number in a parenthesis
means a removal (%)

(12) Operation of the filters of high turbidity influent and capture of SS

1) Experiment method

The drained sludge of the sedimentation tank in the secondary treatment plant was pumped up and added immediately before the screen of the raw water tank of the filter. Turbidity of the raw water was regulated by opening and closing the valve mounted on the discharging side of the pump. The operation was simultaneously performed for the upflow filter and the tri-media filter at the constant filter rate (about 200 m/day). The backwash procedures were that shown in Fig. 3.1.21 plus the final water drainage step with regard to the upflow filter and that shown in Fig. 3.1.23 with regard to the tri-media filter. The filtration was continued till the head loss rises sufficiently. The correlation of SS with high turbidity is as shown in Fig. 3.1.41 and the correlation function and the correlation coefficient are as follows;

$$D = 0.0588\,(SS)^2 + 12209\,(SS) - 0.02616 \qquad \ldots\ldots\ldots\ldots\ldots \qquad (1)$$

$$(r = 0.7876)$$

$$SS = 1.140066 D^{1.03725} \times 10^{-0.003120} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \qquad (2)$$

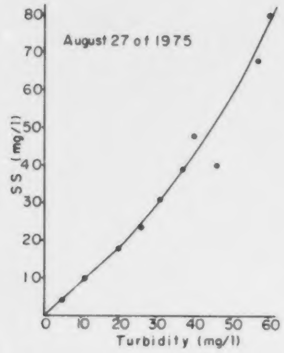
$$(r = 0.9959)$$

where,

SS : SS concentration (mg/ℓ)
D : Turbidity (mg/ℓ)
r : Correlation coefficient

SS was measured by using 5 μm millipore filter mounted on a sintered glass filter according to the sewage examination standard method. Turbidity was measured with the turbidity meter of falling type of Swiss Sigrist company (UP 52-TJ). Calibration of this turbidity meter was conducted by means of silicon dioxide (SiO_2 1 mg/ℓ = 1 ppm). Calculation of SS removal was executed based on Fig. 3.1.41.

Fig. 3.1.41 A Relationship between Turbidity and SS when the Sludge withdrawn from the Secondary Sedimentation Tank is Mixed.



2) Experiment result

Experiment was performed as indicated in Table 3.1.14 and the hourly variation of SS removal is shown in Fig. 3.1.42 and Fig. 3.1.43.

Table 3.1.14 SS Removal Experiment at High Turbidity Influent

Experiment Name	Filter Name	Start Date & Time	Average Filter Rate (m/day)	Filter Run Time (hr:min)	Terminal Head Loss(m)	Influent Average Turbidity (mg/l)	Effluent Average Turbidity (mg/l)	SS Captured (kg/m²)
U-1.1	U-1	75 9/4 11:32	195.4	15:13	2.23	30.4	1.00	3.53
U-1.2	U-1	75 9/5 17:34	195.1	11:56	2.42	46.4	1.91	4.91
U-1.3	U-2	75 9/5 4:16	201.0	13:19	2.70	34.1	1.20	3.85
U-1.4	U-2	75 9/5 18:27	199.4	11:10	2.70	47.2	1.12	4.94
U-1.5	U-2	75 9/6 6:30	196.6	10:38	2.67	41.8	0.60	4.27
D-1.1	D-1	75 9/6 0:51	196.7	9:16	3.00	46.6	3.17	3.78
D-1.2	D-2	75 9/5 21:49	197.1	9:00	2.70	50.3	1.58	3.64
D-1.3	D-1	75 9/6 10:26	197.5	8:27	2.70	39.2	1.62	2.92
D-1.4	D-2	75 9/5 1:05	198.0	9:08	2.85	37.4	1.09	2.93
D-1.5	D-2	75 9/6 7:08	198.1	8:37	2.70	42.4	1.04	3.26
D-1.6	D-2	75 9/5 10:32	196.9	10:59	2.60	42.4	1.75	3.77

Note U-1 Upflow Filter No. 1
U-2 " " No 2
D-1 Tri-Media Filter No 1
D-2 " No 2

Fig. 3.1.42 SS Removal Variation of the Upflow Filter

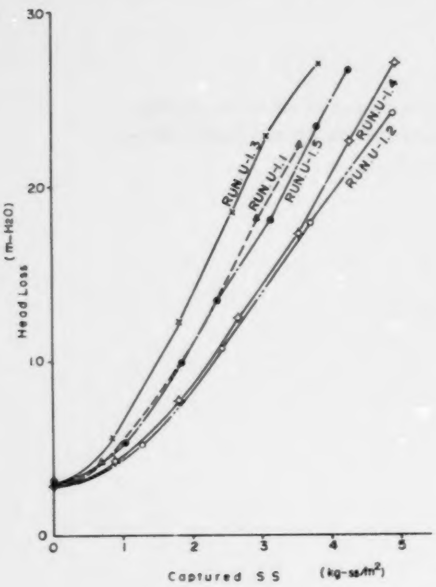
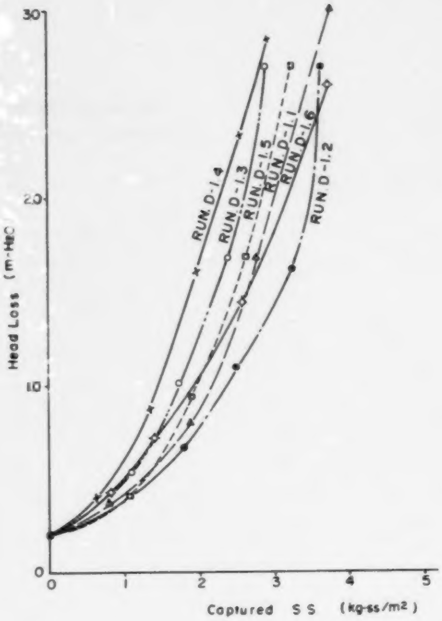


Fig. 3.1.43 SS Removal Variation of the Tri-Media Filter



3) Consiederation

i) Amount of SS captured

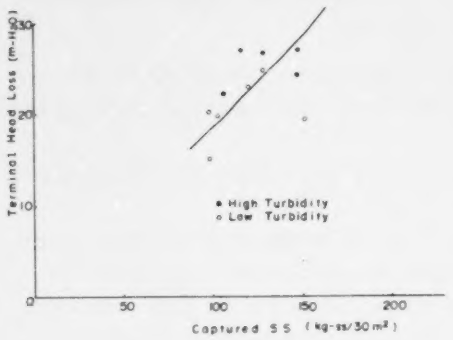
The amount of removed SS at the time of low turbidity (average influent turbidity; 17.5 mg/ℓ) and that at the time of high turbidity (average influent turbidity; 40 mg/ℓ) were almost equal as for the upflow filter as indicated in Fig.3.1.44.

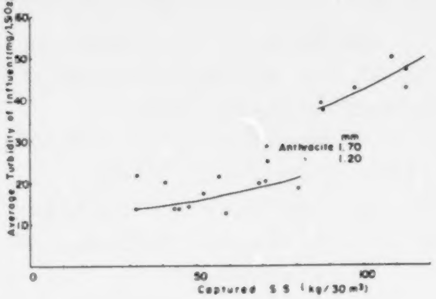
The value is 120 ~ 150 kg/30 m² (4 ~ 5 kg/m²) (0.8 ~ 1.0 lb/SF) in the terminal head loss of 2.70 m.

With regard to the tri-media filter, while the SS removal amount was 30 ~ 70 kg/30 m² (1.0 ~ 2.3 kg/m²) (0.2 ~ 0.5 lb/SF) at the time of low turbidity (anthracite particle size; 1.22 mm), the removal amount at the time of high turbidity (particle size; 1.70 mm) increased up to 90 ~ 110 kg/30 m² (3.0 ~ 3.7 kg/m²) (0.6 ~ 0.8 lb/SF), as shown in Fig. 3.1.45. The head loss distribution comparison in the case of anthracite size; 1.70 mm and 1.22 mm of Fig. 3.1.40 indicates that the increase of head loss was limited to the anthracite layer when the particle size was 1.22 mm, but the head loss increase reaches the sand layer when it was 2.70 mm. Therefore, in the filter media of the anthracite particle size of 1.22 mm, SS removal was small and in the case of the particle size of 1.70 mm, the anthracite layer and the sand layer works effectively for SS removal so that the larger part of SS was removed. Amount of SS removal of tri-media filter is rather inferior to that of upflow filter: about 70 ~ 80%.

Fig. 3.1.44 SS Removal of the Upflow Filter (200 m/day)

Fig. 3.1.45 SS Removal of the Tri-Media Filter (200 m/day)

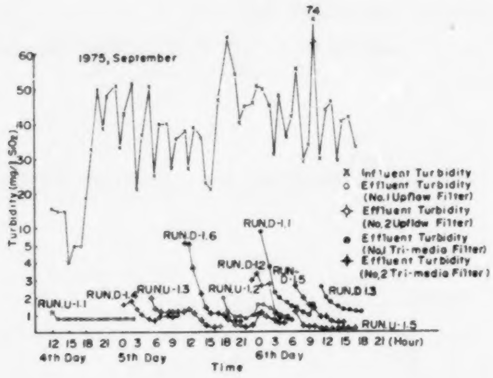




ii) Effluent turbidity

Fig. 3.1.46 presents the turbidity of influent and effluent investigated every one hour. The turbidity of effluent from the both filters were higher at the start of filtration than those of later stage of the runs. The degrees of this deterioration of effluent turbidity were greater in the tri-media filter and also the time required for recovering the stable turbidity is longer in the tri-media filter.

Fig. 3.1.46 Turbidity of Filtrate to the Influent Containing High Turbidity (Filter Rate 200 m/day)



iii) Filter bed condition

No variation was observed in the outer appearance of the filter bed against the high turbidity influent as to the upflow filter. In this experiment six tests were executed and the head loss was augmented to above 1.70 m, but no break of the bed occurred during these runs.

In the tri-media filter, however, SS accumulated on the anthracite surface invades along the wall face and the filter bed condition deteriorates.

Since the general washing method could not recover the original condition of the bed and the following operations became necessary.

i. The backwash water containing chlorine of high concentration (sodium hypochlorite 500 mg/ℓ) was sent to the filter and the chlorinated water was stored to the level right below the trough.

ii. Air blow is performed. The chlorine remaining in the filter bed disappears in 15 min. of continuous air blow.

iii. The water staying in the upper layer of anthracite was lowered down to the filter bed by opening the drainage valve and again the air blow was performed.

iv. SS dissociated from the filter bed was washed away by the backwash water.

The operations of i ~ iv are repeated in correspondence to the necessity.

When much SS remained in the tri-media filter bed, the filter could not sufficiently fluidized during its washing and masses of SS accumulated in a certain part of the bed or the surface of anthracite layer piled up. However this extreme deterioration of filter bed condition did not cause any initial head loss increases. This was because the influent passed through the SS accumulated part with of resistance than other parts in the beginning of filtration. Thus, the effluent quality became worse just after the filtration start as above described. It seemed that, after a while, the SS accumulated part became densely packed, and the effluent quality stabilized. Therefore, the best method to find the deterioration of filter bed condition is the observation of surface of anthracite layer after water drainage. When it was difficult to remove SS remained in the filter bed, the hand operation of breaking masses of SS with a bar during the backwash along with the above i ~ iv was also effective. The backwash method executed once a day was changed to that shown in Fig. 3.1.23 so as to treat the high turbidity influent.

(13) Filtration through the upflow filter at the large flow fluctuation

i) Experiment method

Fig. 3.1.47 Wastewater Flow Variation in the Treatment Plant (1974)

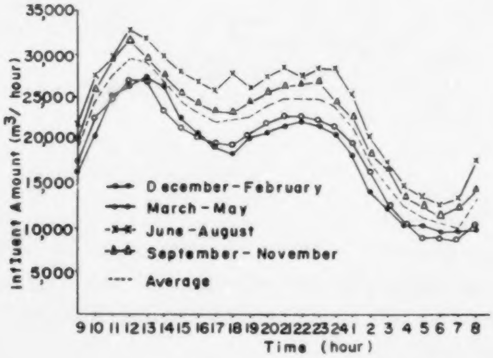


Table 3.1.15 Flow Fluctuation Curve (No. 1)

Hour	Calculated Value		Observed Value	
	Flow Rate (m3/hr)	Filter Rate (m/day)	Flow Rate (m3/hr)	Filter Rate (m/day)
9	356	285	345	276
10	450	360	452	361.6
11	516	412.5	-	-
12	553	4425	560	448
13	544	435	-	-
14	506	405	520	416
15	459	3675	-	-
16	431	345	420	336
17	413	330	412	329.6
18	422	3375	410	328
19	431	345	420	336
20	450	360	452	361.6
21	469	375	480	384
22	469	375	480	384
23	469	375	480	384
24	450	360	452	361.6
1	403	3225	-	-
2	328	2625	306	2448
3	281	225	-	-
4	234	1875	205	164
5	210	168	-	-
6	197	1575	168	1344
7	188	150	150	120
8	244	195	216	1728

The maximum filtration rate was taken at 443 m/day which was maximum filtration rate possible for one filter operation. Applying this rate for the annually averaged maximum flow rate per hour of a flow fluctuation pattern in a certain sewage treatment plant shown in Fig. 3.1.47, the flow rate of the other time was determined in correspondence with this pattern. Thus, the supposed value of flow fluctuation curve (No. 1) in Table 3.1.15 was obtained. The ratio of the maximum to the minimum flow was 3.7 : 1. The control of flow rate against the supposed value was executed by changing the water level of the raw water conduit. The water level change of the raw water conduit was performed with changing the setting of influent flow regulator sending the signal to the influent flow control valve by remote control. The signals to this regulator was sent by the flow indicator which could send 96 types of analogue signals in 24 hours. Accordingly, the signal was cascade changed once 15 minutes. The water level is determined on the following equation;

$$Q = C \cdot a\sqrt{2g(H-Ho)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

where,

a : Bellmouth sectional area 0.0707 m^2 (ϕ 300 m/m)
C : Flow coefficient
Ho : Bellmouth height (height above the bottom of influent conduit)
H : Water level of influent conduit
g : Gravity acceleration
Q : Flow (m^3/sec.)

Based on the water levels (H_1, H_2) in the two flows (Q_1, Q_2) different to each other, C and Ho of each filters determined as follows;

	No. 1 filter	No. 2 filter
C	0.7725	0.7620
Ho	0.203 m	7.196 m

Flow was measured by electro-magnetic flow meter and water level by water level meter of air purge type.

The experiment of large flow fluctuation was to be executed in No. 2 filter and the enumeration equation of flow was obtained;

$$Q = 0.7725 \times 0.0707 \times 2 \times 9.8 \, (H\text{-}7.196)$$

$$H = 7.196 + \left(\frac{Q}{0.2385}\right)^2 \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (4)$$

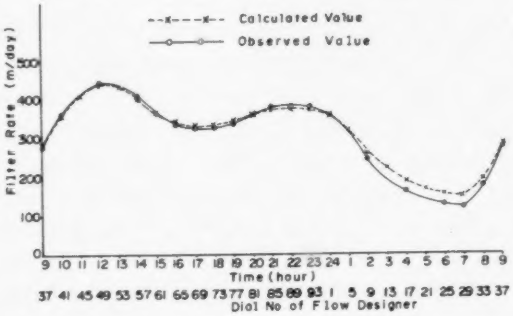
When the unit of Q is m^3/H,

$$H = 7.196 + \left(\frac{Q}{858.6}\right)^2 \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (5)$$

The observed values in Table 3.1.15 were obtained by calculating the water level for each flows with the equation (5), setting it in the flow indicator and measuring the actual flow.

The experiment was conducted in the flow variation pattern of this observed value. Flow variation pattern of the calculated value and the observed value are presented in Fig. 3.1.48.

Fig. 3.1.48 Large Flow Fluctuation Pattern



2) Experiment result

This experiment was performed totally for ten times as shown in Table 3.1.16. The break phenomena occurred three times of these ten. In RUN-2 and 3 it occurred when the flow increased rapidly at the high head loss. In RUN-4, it occurred when the normal backwash was not performed due to the fault of the pinboard timer. In RUN-5 ~ 7, the head loss did not rise up to above 1.70 m within 24 hours. When the head loss rises above 1.70 m, the stress was observed in the grid and the filter bed got into the break zone condition status. In RUN-8 ~ 10, though the head loss was above 1.70 m, no break occurred because the period of rapid flow increase was applied for the filtration start. Therefore, it was found that the break of the upflow filter occurred when the head loss was above 1.70 m and the flow increased suddenly. Incidentally, the break might be caused also by those instantaneous change of flow rate by means of cascade control. It, however, is now being investigated on permissible range of flow fluctuation. The hourly variation of the head loss, the flow rate and the influent quality for every run is shown in Fig. 3.1.49 through 3.1.53.

Table 3.1.16 Experiment on Large Fluctuation of Flow to Upflow Filter No. 2

Experiment NO.	Date	Filter Run Time (hr:min)	Turbidity(average) Influent (mg/l)	Turbidity(average) Effluent (mg/l)	Max. Head Loss (m)	Observations
1	75.10/21 9:45-10/21 17:30	7 45	31	1	2.50	
2	10/22 18:45-10/22 8:35	13 :50	32	1	2.90	Breakthrough at the Head Loss of 2.90
3	10/22 10:15-10/23 9:45	23 30	10	0	2.46	Breakthrough at the Head Loss of 2.46
4	10/23 14:45-10/24 2:45	12 :00	23	1.3	2.40	Breakthrough at the Head Loss of 2.40 (Incomplete back wash) (Air water mixing wash)
5	10/28 15:00-10/29 15:15	24:15	7	1	0.80	
6	10/29 15:00-10/30 16:00	24 :00	9	1	0.93	
7	10/30 17:00-10/31 17:00	24 :00	12	1	1.20	
8	10/31 18:15-11/1 13:00	18:45	16	1	2.64	
9	11/1 14:00-11/5 16:15	98:15	7	<1	2.61	Nov.1 14:00 ~ Nov.4 09:15 Constant Flow Filtration 300m/Day
(9)′	11/4 9:15-11/5 16:15)	(31:00)	(7)	(<1)	(2.61)	
10	11/5 18:00-11/6 16:20	22:20	17.5	1	above3.00	

Fig. 3.1.49 Experiments on Large Flow Fluctuation No. 1, No. 2

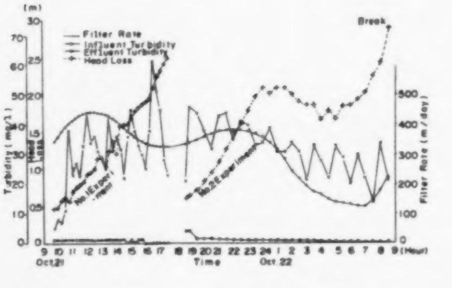


Fig. 3.1.50 Experiment on Large Flow Fluctuation No. 3

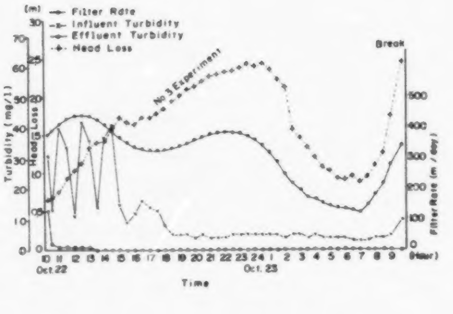


Fig. 3.1.51 Experiment on Large Flow Fluctuation No. 4 (Incomplete Washing)

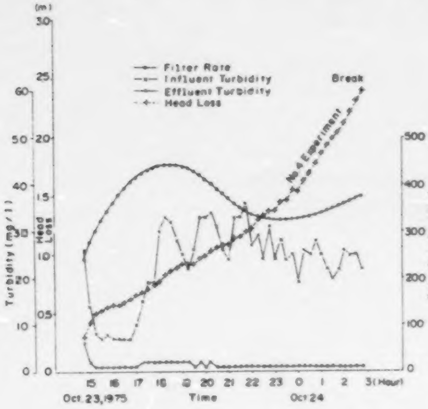


Fig. 3.1.52 Experiment on Large Flow Fluctuation No. 8

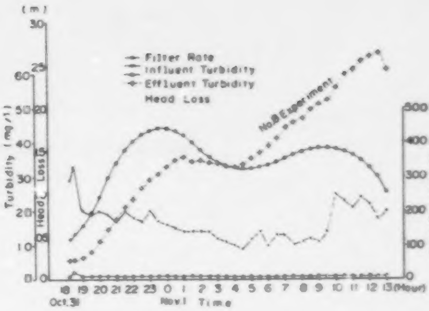
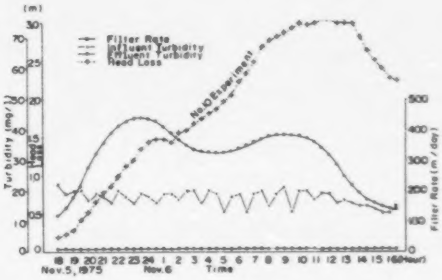


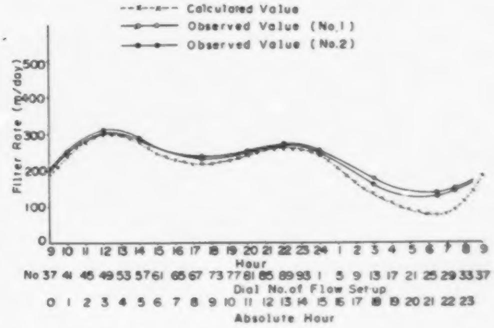
Fig. 3.1.53 Experiment on Large Flow Fluctuation No. 10



(14) Filtration through the upflow filter at the small flow fluctuation

1) Experiment method

Fig. 3.1.54 Small Flow Fluctuation Pattern



The maximum flow of 300 m/day (the maximum filter rate of two filters operation) was applied for the peak flow of Fig. 3.1.47 and the filter rate of each time zone was obtained in a similar manner as in the foregoing experiment and presented in Fig. 3.1.54. A part of the filter rate could not lower as calculated because of the limitation from facility construction and resulting in ratio between the maximum and the minimum value in the actual flow rate of 2.4 : 1 (No. 1 upflow filter) and 2.3 : 1 (No. 2 upflow filter).

2) Experiment result

With regard to the low turbidity influent, 20 times of the experiment were performed for No. 1 upflow filter and 24 times for No. 2 upflow filter, as shown in Table 3.1.17 and 3.1.18. Some hourly variations of them were presented in Fig. 3.1.55 ~ 3.1.57. These results prove that when the head loss is below 1.7 m, breaks does not occur in spite of the variation of the flow velocity.

Table 3.1.17 Experiment on Small Fluctuation of Flow to Upflow Filter No. 1

Experiment NO.	Date	Filter Run Time (hr:min)	Turbidity(average)		Max. Head Loss (m)	Observations
			Influent (mg/l)	Effluent (mg/l)		
1	81 1/21,18:53-1/23,16:22	43:22	9	2	0.75	
2	1/23,16:53-1/24,22:17	29:24	4	1	0.58	
3	1/24,23:08-1/26, 4:33	29:25	3	1	0.67	
4	1/26, 5:23-1/27,10:45	29:22	2	1	0.68	
5	1/27,11:35-1/28,17:09	29:34	3	1	0.61	
6	1/28,17:59-1/29,23:22	29:23	3	1	0.58	
7	1/30,0:13-1/31, 5:46	29:33	4	1	0.67	
8	1/31,6:36-2/1,12:00	29:24	3	1	0.62	
9	2/1,12:50-2/2,18:15	29:25	4	1	0.61	
10	2/2,19:06-2/4, 0:41	29:35	4	1	0.61	
11	2/4,1:31-2/5, 7:07	29:36	5	1	0.66	
12	2/5,7:57-2/6,13:30	29:33	7	2	0.69	
13	2/6,14:21-2/7,19:42	29:21	5	1	0.58	
14	2/7,20:32-2/9, 1:54	29:22	6	1	0.64	
15	2/9,2:44-2/10,8:08	29:24	5	1	0.62	
16	2/10,8:59-2/11,14:23	29:24	6	1	0.61	
17	2/11,15:13-2/12,20:38	29:25	6	1	0.59	
18	2/12,21:28-2/14, 3:02	29:34	7	1	0.67	
19	2/14, 3:35-2/15, 9:27	29:34	6	1	0.63	
20	2/15,10:17-2/16,15:39	29:22	5	1	0.59	
21	4/9,16:05-4/10, 6:38	14:33	20	2	1.90	Break Occurred

Table 3.1.18 Experiment on Small Fluctuation of Flow to Upflow Filter No. 2

Experiment NO	Date	Filter Run Time (hr:min)	Turbidity(average)		Max. Head Loss (m)	Observations
			Influent (mg/l)	Effluent (mg/l)		
1	81 1/21,17:04-1/23,20:45	49:54	9	5	0.90	
2	1/23,21:50-1/25, 2:45	28:55	5	4	0.79	
3	1/25,3:59-1/26, 8:30	28:46	3	2	0.83	
4	1/26,9:27-1/27,14:25	28:58	2	2	0.70	
5	1/27,15:17-1/28,20:13	28:56	3	2	0.68	
6	1/28,21:05-1/30, 3:40	30:35	3	2	0.78	
7	1/30, 4:32-1/31, 9:32	29:00	3	3	0.83	
8	1/31,10:23-2/1, 15:12	28:49	3	3	0.69	
9	2/1,16:03-2/2, 20:53	28:50	4	3	0.65	
10	2/2,21:44-2/4, 2:45	29:01	4	3	0.75	
11	2/4, 3:36-2/5, 7:58	28:22	5	3	0.73	
12	2/5, 8:49-2/6, 4:36	19:47	8	4	0.77	
13	2/6, 5:27-2/7, 1:16	19:49	5	2	0.67	
14	2/7, 2:08-2/7, 21:51	19:43	5	2	0.61	
15	2/7,22:42-2/8, 18:34	19:52	6	2	0.60	
16	2/8,19:25-2/9, 15:07	19:42	5	2	0.63	
17	2/9,15:59-2/10, 11:49	19:50	6	2	0.62	
18	2/10,12:40-2/11, 8:26	19:46	6	2	0.64	
19	2/11,9:17-2/12, 5:04	19:47	6	1	0.66	
20	2/12,5:56-2/13, 1:44	19:48	6	2	0.66	
21	2/13,2:36-2/13, 22:18	19:42	6	2	0.57	
22	2/13,23:09-2/14, 19:00	19:51	6	2	0.59	
23	2/14,19:51-2/15, 15:34	19:43	6	2	0.60	
24	2/15,16:25-2/16, 12:15	19:50	5	2	0.61	
25	4/8,20:30-4/9, 11:35	15:05	21	3.5	2.70	

Fig. 3.1.55 Experiment on Small Flow Fluctuation (No. 2 Upflow Filter, Low Turbidity)

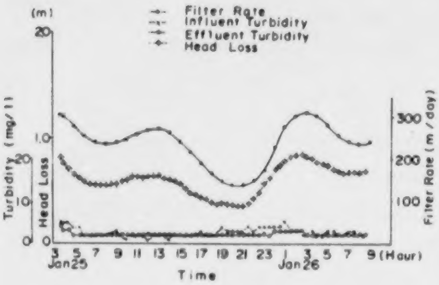


Fig. 3.1.56 Experiment on Small Flow Fluctuation (No. 1 Upflow Filter, Middle Turbidity)

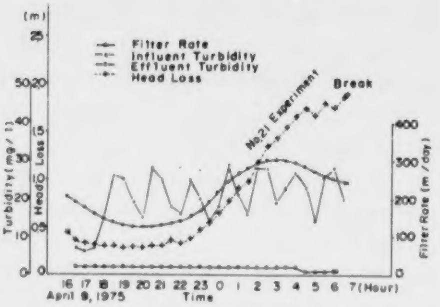
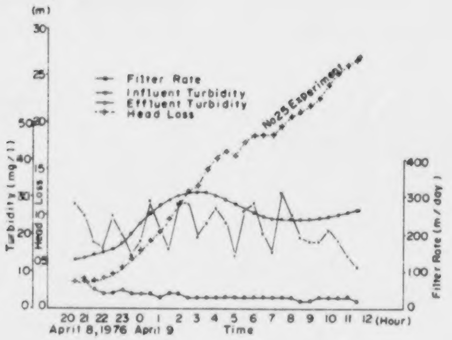


Fig. 3.1.57 Experiment on Small Flow Fluctuation (No. 2 Upflow Filter, Middle Turbidity)



(15) Generation of chironomus

Chironomus generation is one of problems when the chlorination of raw water is not performed.

In the filter basin of Morigasaki Treatment Plant, the generation of chironomus was experienced in August, three months after the operation of the filters started. Chironomus generation, whose nits came from the secondary clarifier, took place in the storage tank of the secondary effluent, because of its slow water current. Then, every open part of the storage tank was sealed not to chironomus fly out. The larvae of chironomus in the influent, however, entered the both filter basins through the feed pump.

The upflow filter has an advantage that the raw water containing chironomus larvae is filtered before they have a contact with atmosphere, so that no chironomus generation is found in the upflow filter. In the tri-media filter, the raw water has a contact with atmosphere before filtered. Therefore, it is difficult to prevent chironomus generation in the tri-media filter.

(16) Comparison of turbidity meters for automatic measurement

1) Experiment method

The following three types of turbidity meters were employed in this experiment;

1. Turbidity meter, Hokushin Electric Works (N), W301-WLS301 (Open liquid surface scattered light method)
2. Ultrasonic washing type turbidity transmitter, Yokokawa Electrics (Y), TYPE 8562 (Scattered light and permeable light operation system)
3. Turbidity meter, Swiss Sigrist (S), UP52, (permeable light and standard light comparison system)

Secondary settled sludge was mixed into the secondary effluent (SS 10 mg/ℓ) from March 3 till April 10 of 1976 so as to increase the turbidity and then the indicated values by the above three turbidity meters were compared. Sludge was added four times on March 3, April 13, April 5 and April 9 ~ 10 and the each turbidities are indicated in Fig. 3.1.58 ~ 3.1.62. The indicated value of each turbidity meters in the secondary effluent are indicated in Fig. 3.1.63.

Fig. 3.1.58 Comparison of Turbidity Meter (High Turbidity)

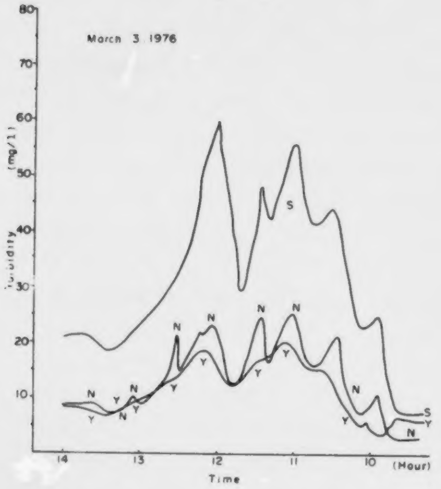


Fig. 3.1.59 Comparison of Turbidity Meter (High Turbidity)

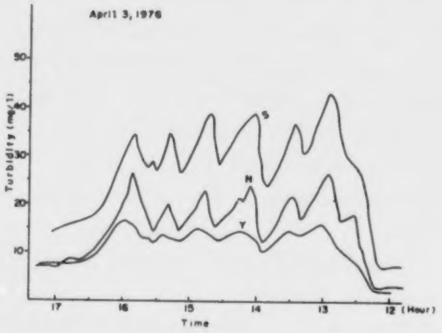


Fig. 3.1.60 Comparison of Turbidity Meter (High Turbidity)

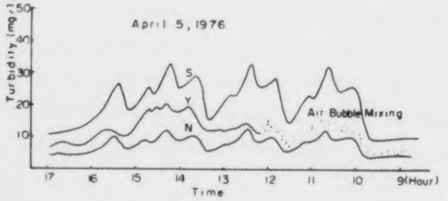


Fig. 3.1.61 Comparison of Turbidity Meter (Fluctuation of N Company Base)

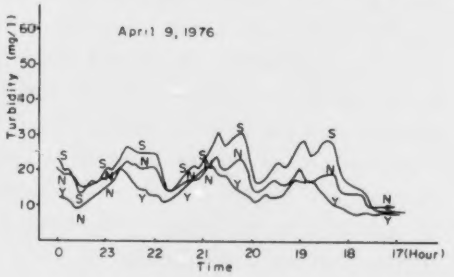


Fig. 3.1.62 Comparison of Turbidity Meter (Fluctuation of N Company Base)

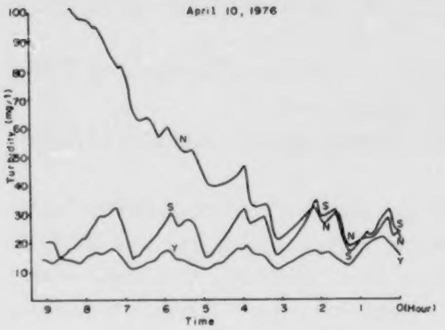
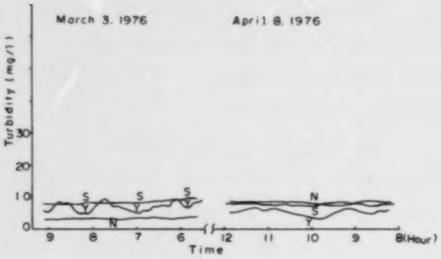


Fig. 3.1.63 Comparison of Turbidity Meter (Low Turbidity)



2) Experiment result

i) The indicated value of N and S are stable at the time of low turbidity (below 10 mg/ℓ). The indicated value of Y has an irregular fluctuation. (Fig. 3.1.63)

ii) S and Y are excellent in the longtime stability. In case of N, the fluctuation of the base is found in Fig. 3.1.63. This is because SS is accumulated in the bottom of the tube forming the rest water surface, the water depth becomes shallow and irregular reflected lights augment. On April 10, the turbidity of N rose, finally reached 100% and did never return. When SS accumulated at the bottom of the tube was removed by flashing with city water, it became to indicate the normal value.

iii) S was the most excellent, N was the second and Y was the worse in the range of turbidity fluctuation.

(17) Particle size distribution in the effluent

Particle size distribution of SS contained in the secondary effluent and the filtered water was measured by HIAC fine grain meter and shown in Fig. 3.1.64 ~ 3.1.66. Fig. 3.1.64 shows about the filter rate of effluent from the usual secondary treatment plant at the filter rate of 500 m/day with the inlet turbidity of 14 mg/ℓ and Fig. 3.1.65 shows the filtered effluent at the filter rate of 200 m/day which the secondary drained sludge was mixed into the secondary effluent. The number of particles sized below 10 μm was not largely different between the mixed raw wastewater and the secondary effluent, but for the particles of the size above 10 μm the number was much larger in the mixed raw wastewater. In both cases, the number contained in the filtered effluent was smaller than in the influent in the range of the particle size above 7.5 μm. This shows that the particle size of SS removed by filtration is above 7.5 μm. SS above 7.5 μm of the carry-over from the secondary clarifier can also be removed.

Fig. 3.1.64 Particle Size Distribution (Middle Turbidity)

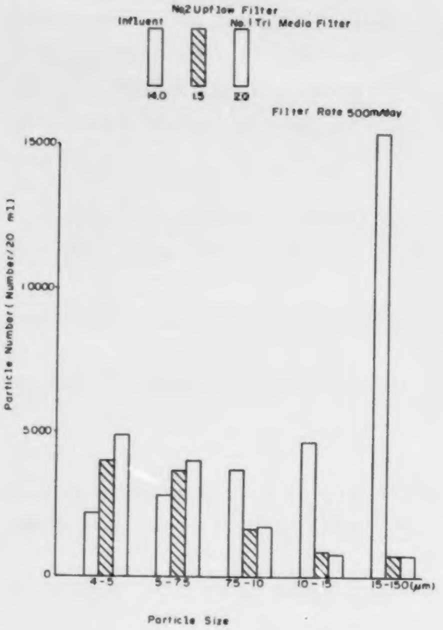


Fig. 3.1.65 Particle Size Distribution (High Turbidity)

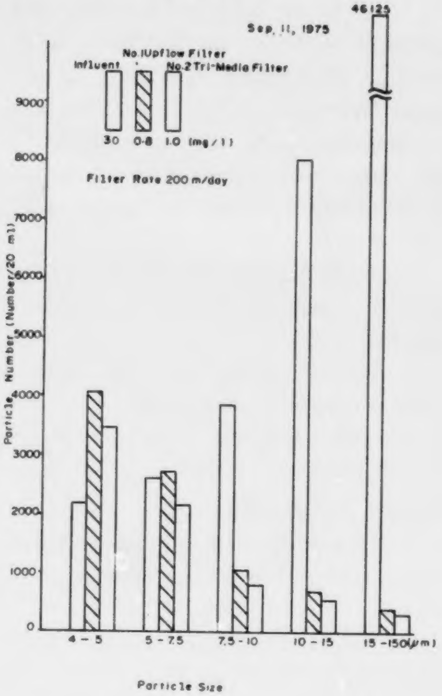
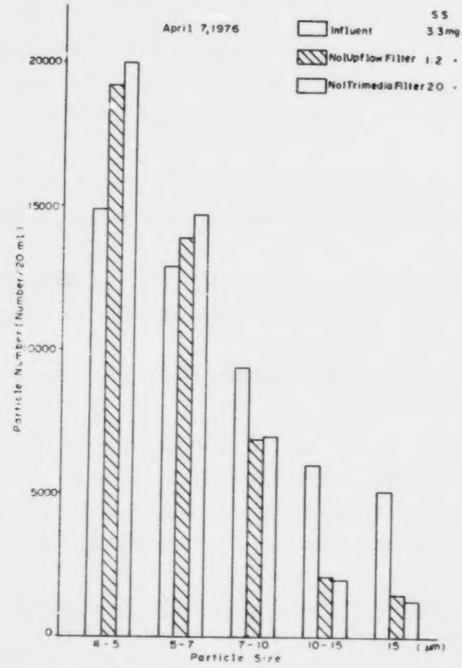


Fig. 3.1.66 Particle Size Distribution (Low Turbidity)



(18) Boring investigation of upflow filter media

1) Method

As to the upflow filter operated continuously for two years after the introduction of water, the boring investigation was performed on the filter media (two places), all the filter sand were carried out and the investigations of the supporting sand layer and the water nozzle were executed. This investigation had the purpose of finding out the quantity of the foreign matters which was not removed by the traveller screen (opening of 6 mm) mounted in the entrance of the raw water tank, entered into the filter and caught in it.

2) Result

i) No foreign matter was found in the filter sand and the sand itself was clear.

ii) The particle size of the filter sand tends to become smaller along with the operation.

iii) The foreign matters (vinyl strips etc.) in the supporting sand layer are slight and tend not to be accumulated.

iv) Choking of water nozzles caused by the foreign matters contained in the influent was not observed.

3.1.3 SUMMARY

Coagulation-sedimentation filtration experiment and the comparison experiment of the upflow filter and the downflow filter have shown summarily the followings;

a. Coagulation-sedimentation filtration method is effective for the removal of phosphorus, BOD and SS.

b. Direct filtration method is sufficient for only BOD and SS removal.

c. Flow proportion control method is enough for the alum addition to perform the coagulation-sedimentation treatment.

d. Increase of the filter rate up to 500 m/day (8.5 GPM/SF) did not have any adverse effect to the effluent quality and also to the systems operation.

e. No difference in the effluent quality was observed between the upflow and the tri-media filter.

f. The average removals of SS and BOD were 90% and 80% respectively.

g. In this study, the optimum backwash procedures have been found for the both filter systems.

h. Amounts of SS removed were about 5 kg/m^2 (1.0 lb/SF) through the upflow filter and about 3.7 kg/m^2 (0.8 lb/SF) through the tri-media filter (anthracite particle size of 1.70 mm).

i. The supporting gravels of the upflow filter effectively functioned as filter media for SS removal.

j. Breakthrough problem in the upflow filter during constant flow filtration was perfectly solved by adopting the adequate washing method.

k. When flows rapidly fluctuate during high head loss stage of filtration runs, sand bed break may occur. However, this can be overcome and prevented by countering operational procedures.

l. The anthracite of about 1.7 mm in its effective size is advantageous to be employed in the tri-media filter.

m. The practical flow rate of the upflow filter was larger than that of the tri-media filter. This realizes that the upflow filter has the larger SS removal capacity from a practical point of view.

n. Removal of E. coli can be expected through filtration.

o. The upflow filter has an advantage that the observation of the effluent quality and that flying out of generated chironomus can be prevented by the filter bed.

p. SS removal at the filtration of high turbidity influent is almost equal to that at low turbidity but the backwash must be carefully conducted.

q. The size of particles that can be removed by filtration is above 7 μm.

r. Choking of water nozzles was not observed in the upflow filter and the supporting sand layer was also kept clear.

3.2 FILTRATION STUDY AT THE KYOTO PILOT PLANT

The followings are a summary on the suspended solid removal by filter and carbon contractor at the Kyoto pilot plant.

The section – 3.1.1 – deals with the solid removal by filter of secondary effluent and alum precipitated effluent under varying influent flow. The sewage inflow into a treatment plant varies every moment. Its variation is extremely great from a low in the night on dry weather to a high on storm weather.

There are two possible ways to meet the inflow variation – one, the constant filtration with flow equalization ponds and the other, the varying filtration allowing the influent flow variation. Because of a omission of a flow equalization pond, the varying filtration is superior to the constant filtration, if there is no great difference as to the quality of effluent and the suspended solid loading between the two. This report sums up the results from October, 1975 to March, 1976.

The section – 3.1.2 – is a study on the possibility of removing both organics and solids by granular carbon. The influents for testing here are filtered secondary effluent, secondary effluent and tertiary alum precipitated effluent.

In this method, there are questions as mentioned below.

1) Is the necessary filtration run length expected by carbon layer?

2) Is the suspended solids in the effluent satisfactory?

3) Is there a drop in organic removal efficiency of carbon by accumulated solids on the surface or in the micropore of carbon?

4) Is there a drop in carbon regeneration efficiency by accumulated solids on the surface or in the micropore of carbon?

The feasibility of this method could be judged when the necessary information is obtained. There had been no detailed report on this method in this country. Then the Public Works Research Institute began the study on this method on November 8, 1976, by using carbon contractors at the Kyoto pilot plant.

This report is an interim report summing up operational results in the two month period from November to December, 1976. Some data obtained on the questions, No. 1, 2, 3 is contained in this report. A study on the question No. 4 is planned for February, 1977.

3.2.1 SOLID REMOVAL BY FILTER UNDER VARYING FLOW

a. Experimental Procedure

Table 3.2.1 shows the specifications of the gravity, down-flow type filter used for this experiment at the Kyoto pilot plant. There are two filters of the same specifications, one is for the secondary effluent, and the other is for the effluent from alum precipitation.

Table 3.2.1 Specification of Filter

Filter

Surface Area (m^2)	1.0 x 1.2 = 1.2

Filter Media

Grain Size	Anthracite	E.S. (mm)	1.62
		U.C. (mm)	1.33
	Sand	E.S. (mm)	0.61
		U.C. (mm)	1.26
Depth (mm)	Anthracite	625	
	Sand	375	
	Total	1,000	

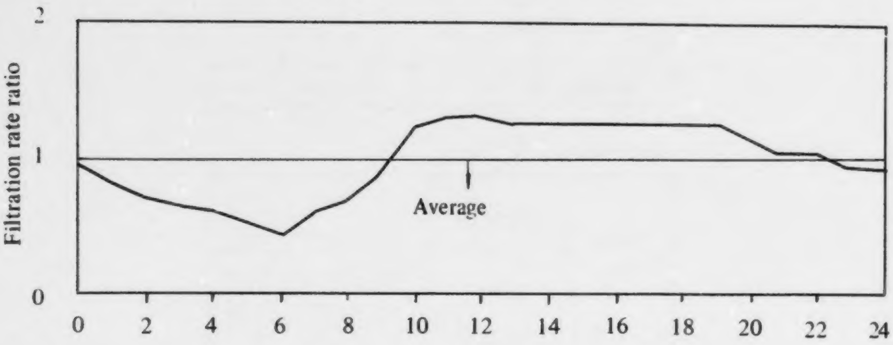
Support Media

Grain Size (mm)	Depth (mm)
19.1 ~ 12.7	50
12.7 ~ 6.73	50
6.73 ~ 3.36	50
3.36 ~ 2.00	50
Total	200

A flow-rate pattern transmitter is set to give flow variation. The circuit for controlling the inflow to the filter consists of two elements. One is the feedforward control of revolution numbers of the roots pump connected to the effluent pipe of the filter. The control works by electric signals from the above mentioned flow-rate pattern transmitter. The other is the feedback control by electric signals from the level meter, which is set on the triangle weir for the effluent flow measuring.

Fig. 3.2.1 shows the flow variation used for this experiment. The variation represents a typical pattern of the inflow into the Toba Sewage Treatment Plant where the pilot plant is located.

Fig. 3.2.1 Flow Variation



Time	Filtration rate ratio	Time	Filtration rate ratio	Time	Filtration rate ratio	Time	Filtration rate ratio
1	0.832	7	0.616	13	1.281	19	1.248
2	0.731	8	0.721	14	1.278	20	1.147
3	0.668	9	0.904	15	1.275	21	1.058
4	0.606	10	1.242	16	1.294	22	1.058
5	0.534	11	1.301	17	1.265	23	0.983
6	0.459	12	1.301	18	1.268	24	0.960

The filtration is terminated when the total head loss reaches 3 meters.

The turbidity in both influent and effluent was measured continuously by the surface-scatter type turbidimeter and then the turbidity was converted into suspended solids through the relation of suspended solids to turbidity shown in Fig. 3.2.2 and Fig. 3.2.3.

Fig. 3.2.2 Turbidity ~ SS (Secondary Effluent and Its Filter Effluent)

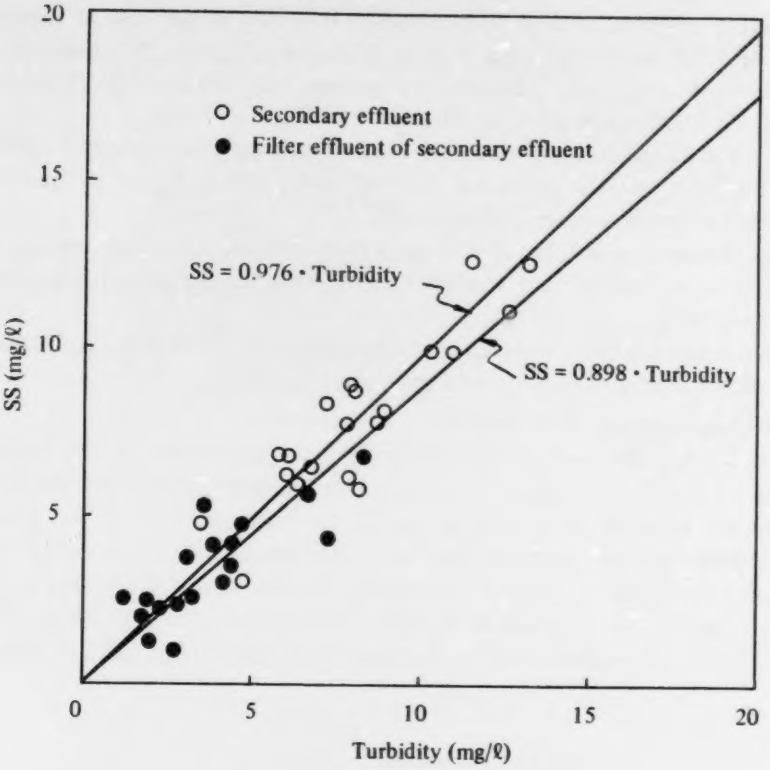
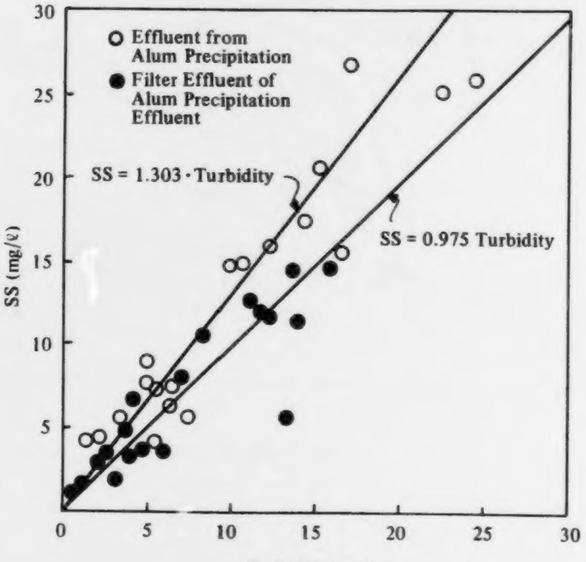


Fig. 3.2.3 Turbidity ~ SS (Alum Precipitation Effluent and Its Filter Effluent



In discussing the experimental results, suspended solids are calculated from an average turbidity in every one hour from the beginning to the end of each case of filtration (a weighted average with filtration flow rate in the case of varying filtration). And the suspended solid loading is expressed by [g·SS captured/m^2· sectional area of filter] and obtained by average suspended solids in influent and effluent, filtration run length and filtration flow rate.

The total head loss was measured continuously by the level meter and the head loss in the filter bed was measured at 10:00 and 16:00 at the depths of 10, 30, 50, 75, 100 cm from the surface of filter media.

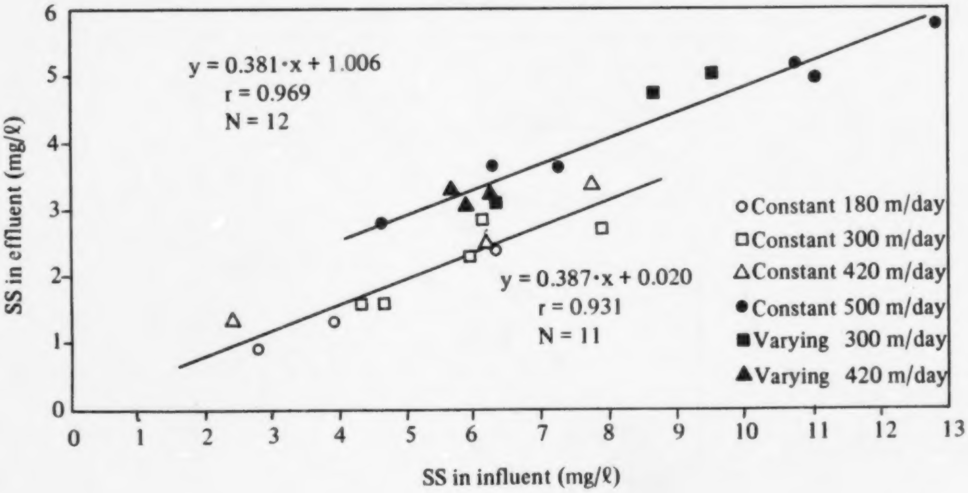
The washing of a filter consisted of nine minutes of surface washing (0.2 m/minute), nine minutes of back washing (1.03 m/minute) and seven minute overlap of the two washings.

The expansion ratio of media by backwashing was set at 20%. And the washing water volume needed for above condition was 11.2 m^3.

b. Filtration of Secondary Effluent

Fig. 3.2.4 is the relationship between suspended solids in the influent and effluent. It shows that suspended solids in the effluent increase with the increase of those in the influent. Also seen in the figure is the presence of two groups with distinct difference in filtration flow rates. Group-A is of the rates of 180, 300, 420 m/day by constant filtration. Group-B is of the rate of 500 m/day by constant filtration and of the average rates by varying filtration of 300 m/day (flow rate range, 138 ~ 390 m/day) and 420 m/day (flow rate range, 193 ~ 546 m/day).

Fig. 3.2.4 SS Removal by Filter
(Influent: Secondary Effluent)



When the relationship of suspended solids in the influent and the effluent in each group is assumed linear, the regression lines obtained are:

Group-A [SS in effluent (mg/ℓ)] = 0.387 x [SS in influent (mg/ℓ)] + 0.020 . . (1)
Group-B [SS in effluent (mg/ℓ)] = 0.381 x [SS in influent (mg/ℓ)] + 1.006 . . (2)

The SS removal of Group-A obtained by eq. (1) is about 61% regardless of the SS in the influent. The gradient of eq. (2) almost equals to that of eq. (1). However, the intercept of eq. (1) is about zero against about 1 mg/ℓ of eq. (2). This means that the SS in the effluent of Group-B is by about 1 mg/ℓ higher than that of Group-A regardless of the SS in the influent. Accordingly, in Group-B, the SS removal as calcurated by eq. (2) increases with the increase of the SS in the influent, standing at 42, 49, 54% respectively against the SS in the influent of 5, 8, 12 mg/ℓ.

For a comparison between constant filtration and varying filtration, take the filtration flow rate of 300, 420 m/day common to the two methods. In this case, the SS of the effluent by varying filtration is estimated to be higher by about 1 mg/ℓ than that of the effluent by constant filtration when the SS of the influent is the same. Because constant filtration belongs to Group-A and varying filtration to Group-B. However, it must be noted that varying filtration with an average rate of 300 m/day has the maximum rate of 391 m/day. Therefore, when only filtration rates are taken into consideration, it is hard to conclude that the effluent quality by varying filtration is worse than that by constant filtration. It is considered that the true problem lies in the change of filtration flow rate itself.

Fig. 3.2.5 shows the SS loading in the case of the total head loss set at 3 m. The SS loading increases with the increase of the SS in the influent. However, at the flow rate of 500 m/day, this tendency ends when SS in the influent exceed 10 mg/ℓ. The SS loading also differs with the change in filtration flow rates. The readings at the rates of 180, 500 m/day are evidently smaller than those at the rates of 300, 420 m/day. For example, when the SS in the influent is 7.5 mg/ℓ, the SS loading for each of the rates of 180, 300, 420, 500 m/day is about 2,300, 3,500, 3,500 and 2,300 g/m^2 respectively.

Fig. 3.2.5 SS Loading of Filter
(Influent: Secondary Effluent)

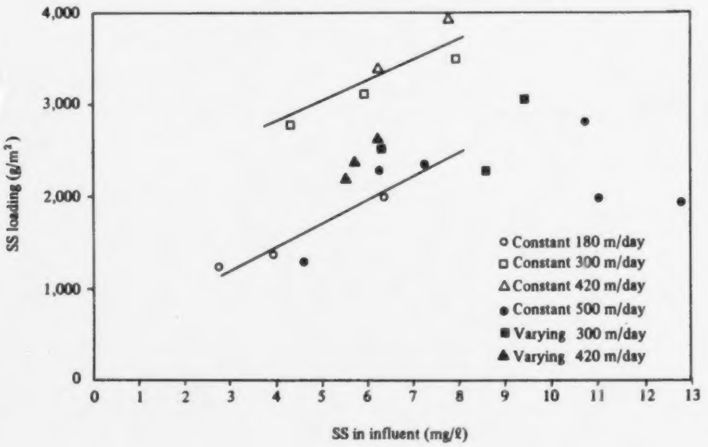


Fig. 3.2.6 shows an example of the relationship between solids in the washing waste and the washing time being obtained by the same procedure mentioned before. (Fig. 3.2.7) The SS loading obtained from Fig. 3.2.6 is 2,700 g/m². This value should agree with the SS loading captured by the filtration. However the former loading exceeds loading about 20% more than the latter one (2,300 g/m²). Only one example is shown here. But more study is planned for turbidity measuring of washing waste.

Fig. 3.2.6 Filter Washing

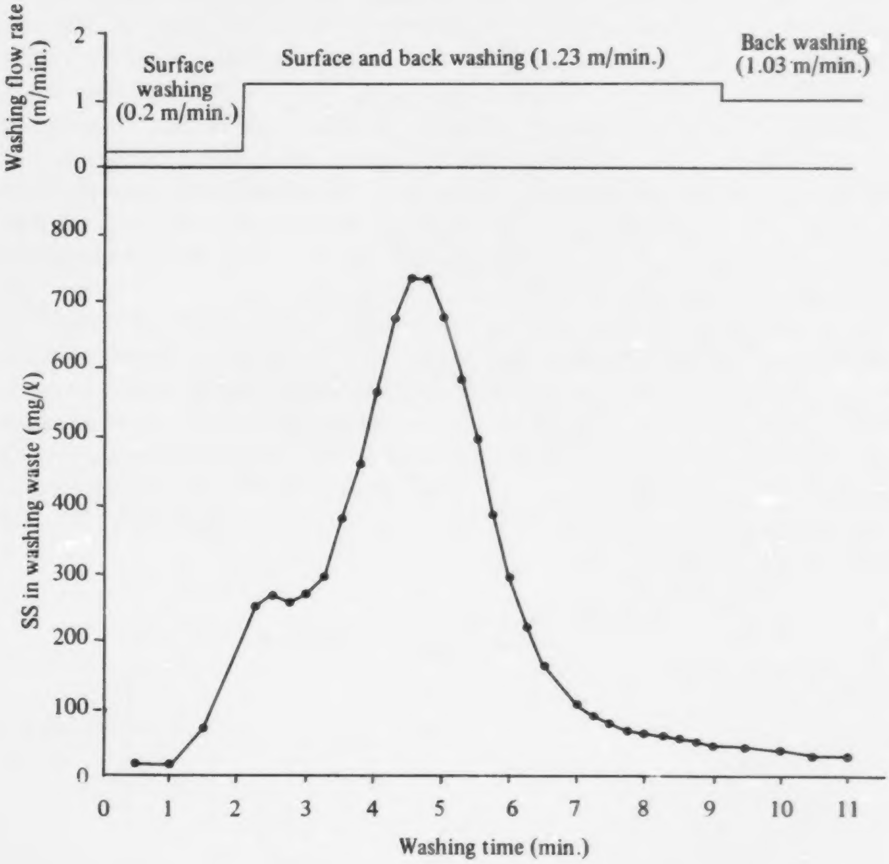


Fig. 3.2.7 Turbidity ~ SS (Filter Washing Waste)

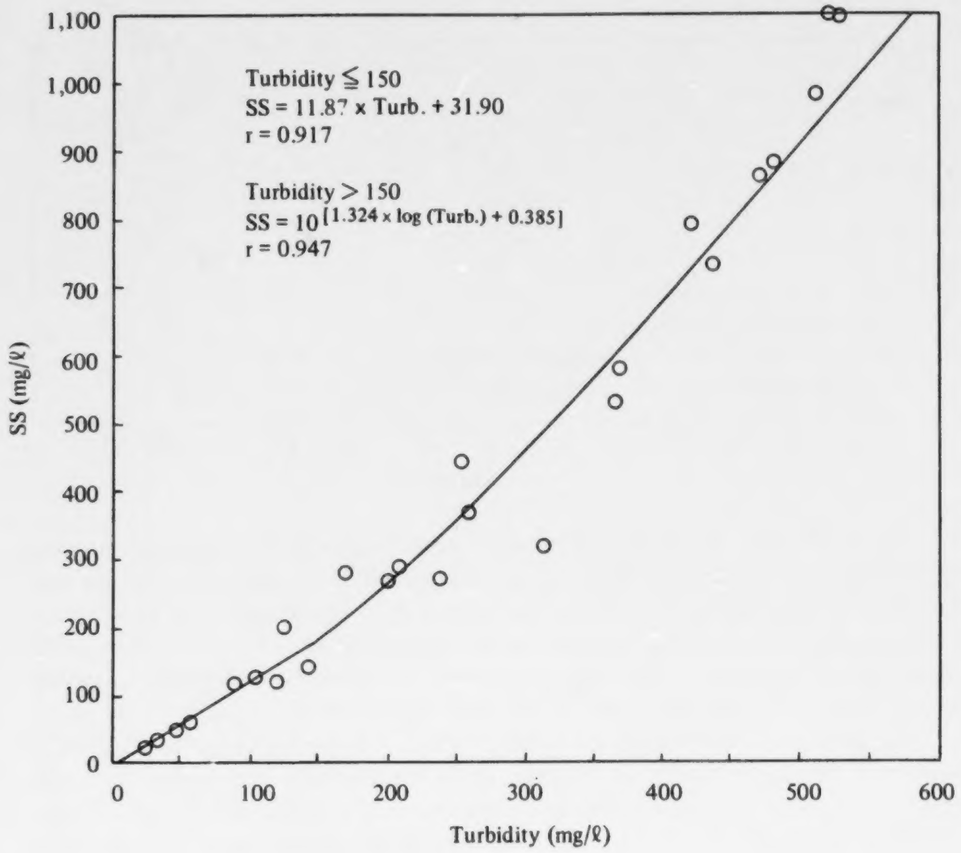


Fig. 3.2.8 is the relationship between suspended solids in the influent and the filtered volume. It shows that the filtered volume by constant filtration and varying filtration is almost the same at the flow rates of 300, 420 m/day. Accordingly, at the rates of 300, 420 m/day, the comparison between the SS loading by constant filtration and that by varying filtration is reduced to the comparison of the amount of SS removed. For example, in the case of 7.5 mg/ℓ of SS in the influent, the amount of SS removed obtained from Fig. 3.2.4, are 4.6 mg/ℓ for constant filtration and 3.6 mg/ℓ for varying filtration. Therefore, the SS loading by varying filtration at the rates of 300, 420 m/day is about 80% of that by constant filtration (3.6/4.6 = 0.78).

Fig. 3.2.8 Filtered Volume
(Influent : Secondary Effluent)

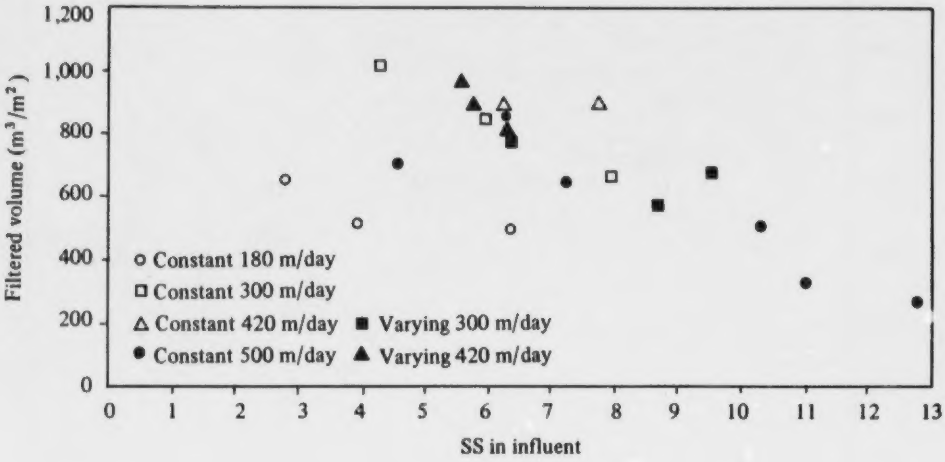
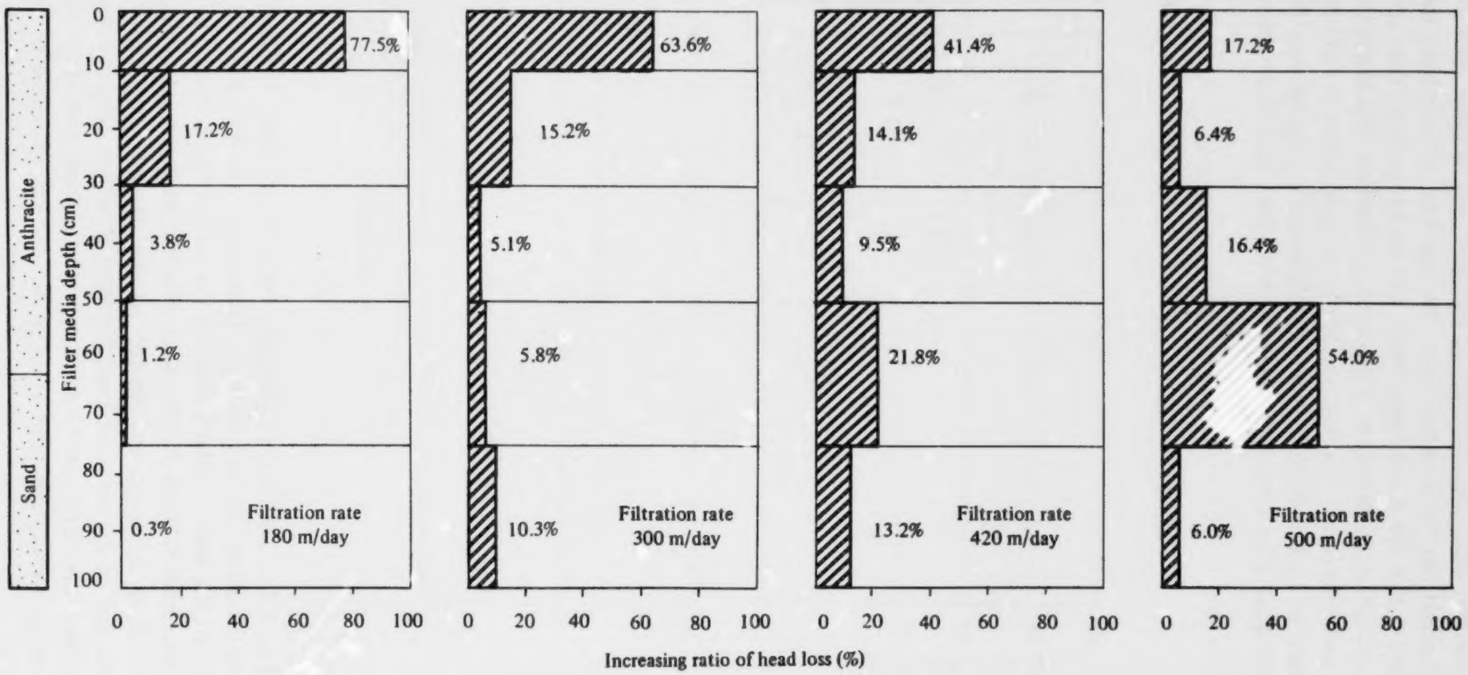


Fig. 3.2.9 gives the relationship between bed depth and the average increase ratio of the head loss in filter beds during constant filtration. The head loss increase ratio here is expressed by the ratio of the value of head loss minus initial head loss at a selected depth of filter bed against the similarly obtained value at the filter media depth of 1,000 mm. The figure shows considerably difference existing in the forms of suspended solids capturing, depending on filtration flow rates.

At the rate of 180 m/day, the form of filtration is the typical surface filtration with the head loss increase ratio standing at 78% in the anthracite surface layer and at a mere 1.5% in the anthracite-sand border and sand layers combined. At the rate of 300 m/day, the head loss increase ratio is still fairly high at 64% in the anthracite surface layer, but the ratios are almost uniform in the layers below. At the flow rate of 420 m/day, the ratio drops to 41% in the anthracite surface layer and the percentage is 22 in the anthracite-sand border, indicating that solids are removed more uniformly in every filter bed layer than in other rates. In the case of 500 m/day, the head loss increase ratio in the anthracite layer becomes small, while the ratio stands at 54% in the anthracite-sand border layer.

The fingings would allow the estimation that filtration of secondary effluent by the filter media composition used this experiment, depends on the surface layer at the rate of 180 m/day, on all layers at the rate of 300 ~ 420 m/day and on the sand layer at the rate of 500 m/day. Because of the reason mentioned above, it seems, that the SS loading is small at the rate of 180 m/day, that the effluent quality is a little bad and SS loading is also small at 500 m/day and that both effluent quality and SS loading are good at the rates of 300 and 420 m/day.

Fig. 3.2.9 Head Loss ~ Filter Media Depth (Influent : Secondary Effluent)



Hour-by-hour measurement of the head loss in filter beds in the varying filtration has not yet been carried out. In this way of filtration, an average speed of 420 m/day has the speed range from 193 to 546 m/day, thus covering all filtration forms by constant filtration – the surface filtration, the in-depth filtration and the sand layer filtration. The hourly change of head losses in the varying filtration will be observed in the future. And the observation may explain the reason why the effluent quality by varying filtration is worse than that by constant filtration.

Fig. 3.2.10 The Time When the Total Head Loss Reached 3 m

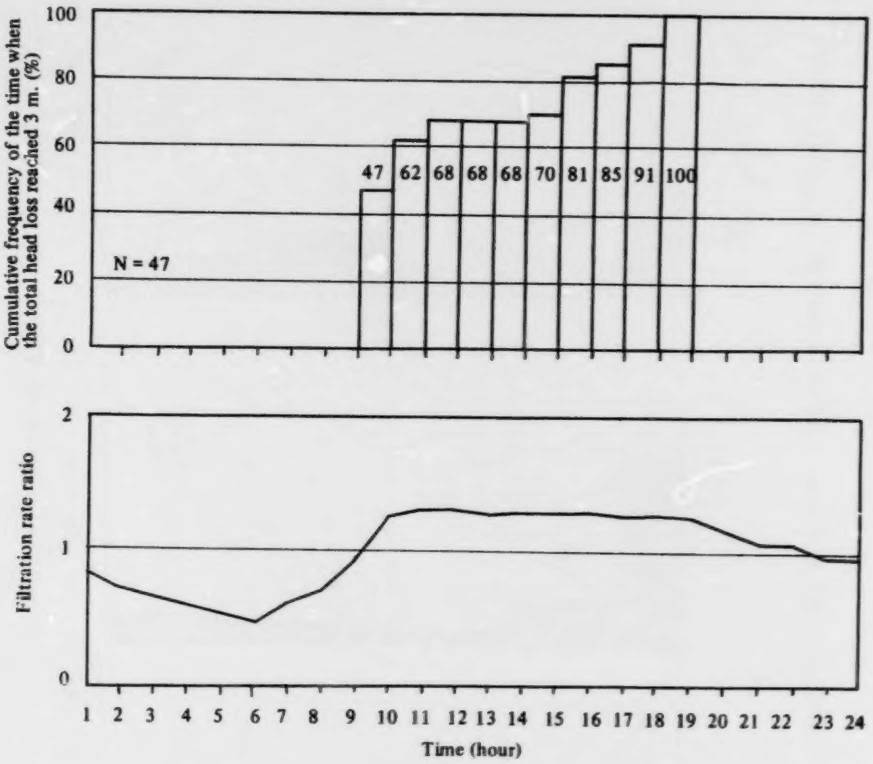
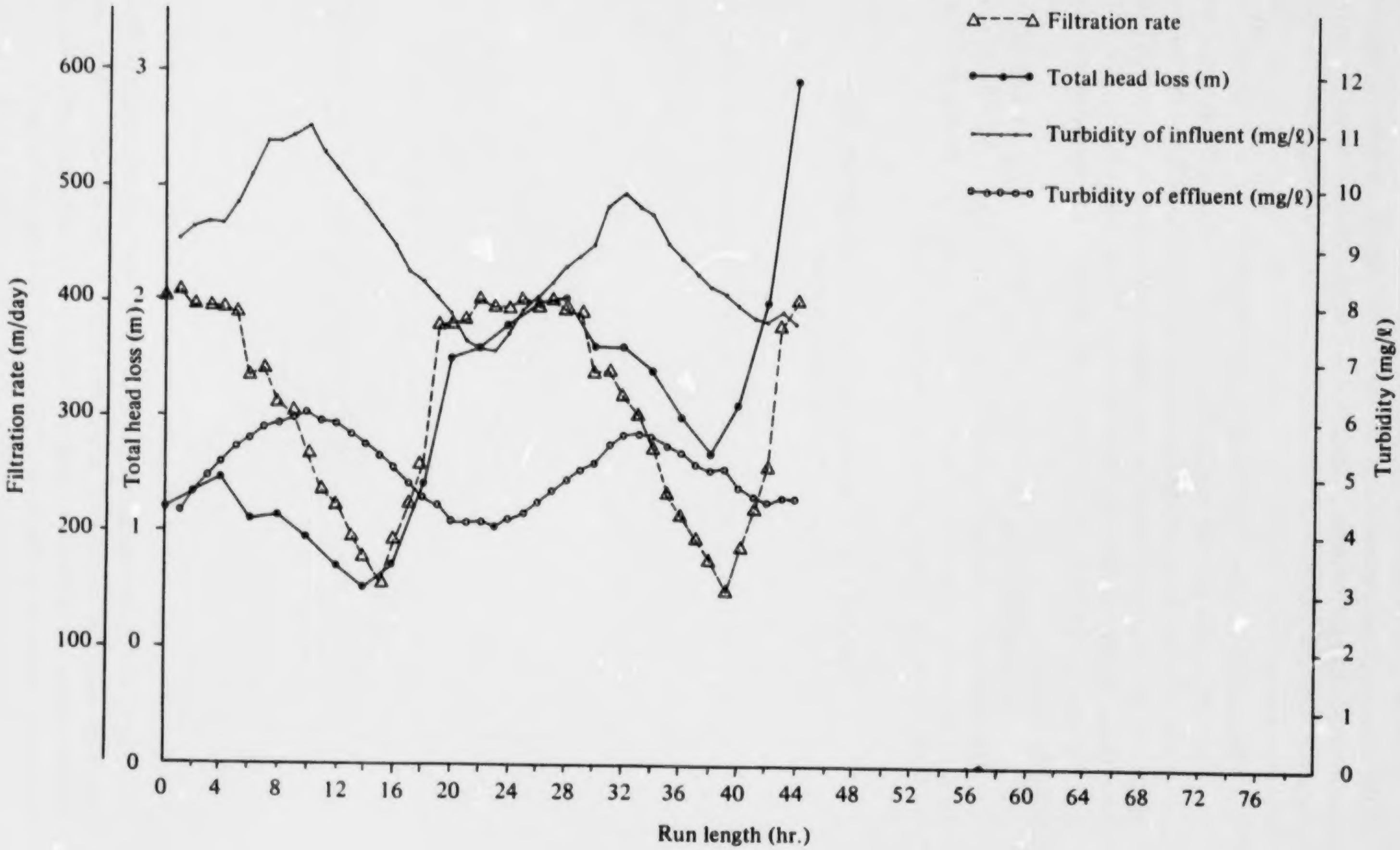


Fig. 3.2.10 shows the rationship between, the time of filtration termination in varying filtration and its cumulative frequency. The data used for this figure are not limited to the results obtained in the period of this experiment. The filtration start time was from 9:00 to 16:00. According to the figure, the times of the total head loss reaching 3 meters are limited to the closing periods of filtration flow rate increase and at the times of high filtration flow rate. Especially noteworthy is that about half the filtration runs stop in the closing period of filtration flow rate increase or within one hour from 9:00. See one example of varying filtration in Fig. 3.2.11.

Fig. 3.2.11 Varying Filtration



The results above indicate the possibility that in varying filtration, filters reach to the terminal head loss almost at the same time, during the time of influent flow rate increase, and the necessary capacity can't be maintained. In such a case, washing of the filters at the time of low influent flow or other preventive means would become necessary.

c. Filtration of Tertiary Alum Precipitated Effluent

Fig. 3.2.12 presents the relationship between suspended solids in the influent and in the effluent. It shows that suspended solids in the effluent increase linearly along with the increase of those in the effluent. And there is no great difference in the effluent quality between constant filtration at the flow rates of 120 ~ 300 m/day and varying filtration at the rate of 180 m/day. The regression line obtained from the figure is:

$$[\text{SS in effluent (mg/}\ell)] = 0.521 \times [\text{SS in influent (mg/}\ell)] - 0.360 \quad \ldots (3)$$

The SS removal obtained through eq. (3) is 66, 60, 55, 52, 50, 49% respectively for each SS in the influent of 2, 3, 5, 10, 20, 30 mg/ℓ. This means that the lower the SS in the influent is, the greater the removal becomes.

Fig. 3.2.12 SS Removal by Filter (Influent : Effluent from Alum Precipitation of Secondary Effluent)

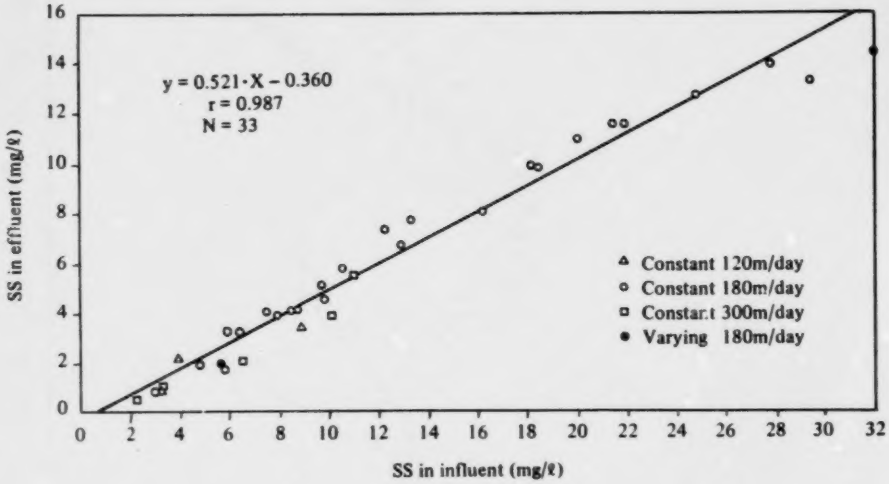


Fig. 3.2.13 shows the SS loading at the total head loss of 3 meters. The loading increases with the increase of suspended solids in the influent. There also is no evident difference in the SS loading between constant filtration at the rates of 120 ~ 300 m/day and varying filtration at 180 m/day. The regression line is:

$$[\text{SS loading (g/m}^2)] = 66 \times [\text{SS in influent (mg/}\ell)] + 320 \ldots\ldots\ldots\ldots (4)$$

Fig. 3.2.13 SS Loading of Filter (Influent : Effluent from Alum Precipitation of Secondary Effluent)

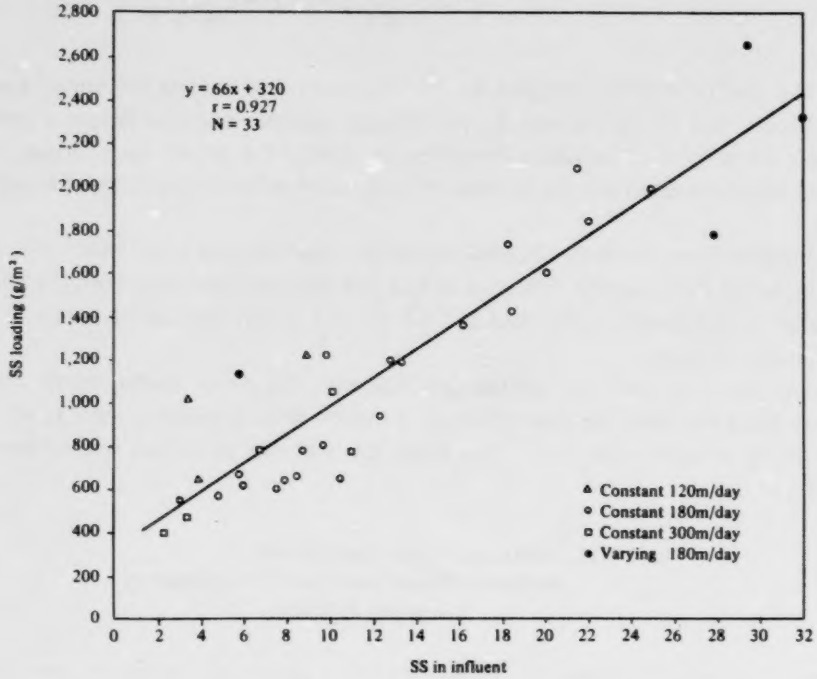
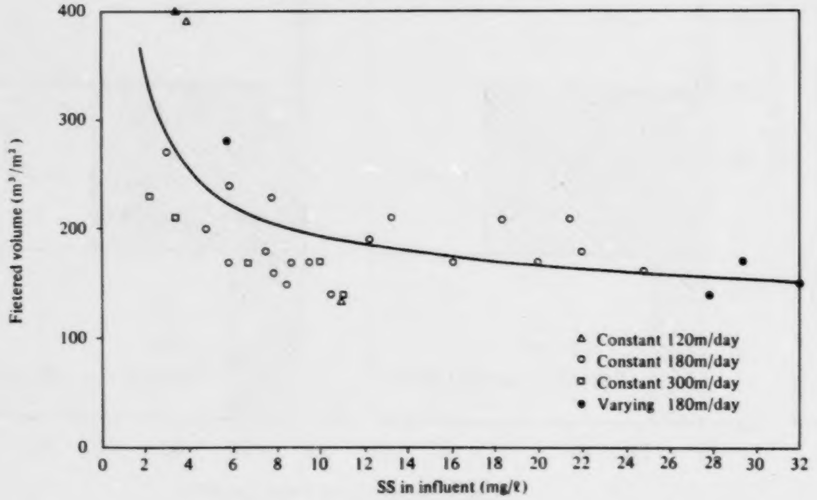


Fig. 3.2.14 is the relationship between suspended solids in the influent and the treated volume at the total head loss of 3 meters. The curve in the figure is calcurated through eq. (3) and eq. (4) in accordance with the following equation.

Fig. 3.2.14 Filtered Volume (Influent : Effluent from Alum Precipitation of Secondary Effluent)



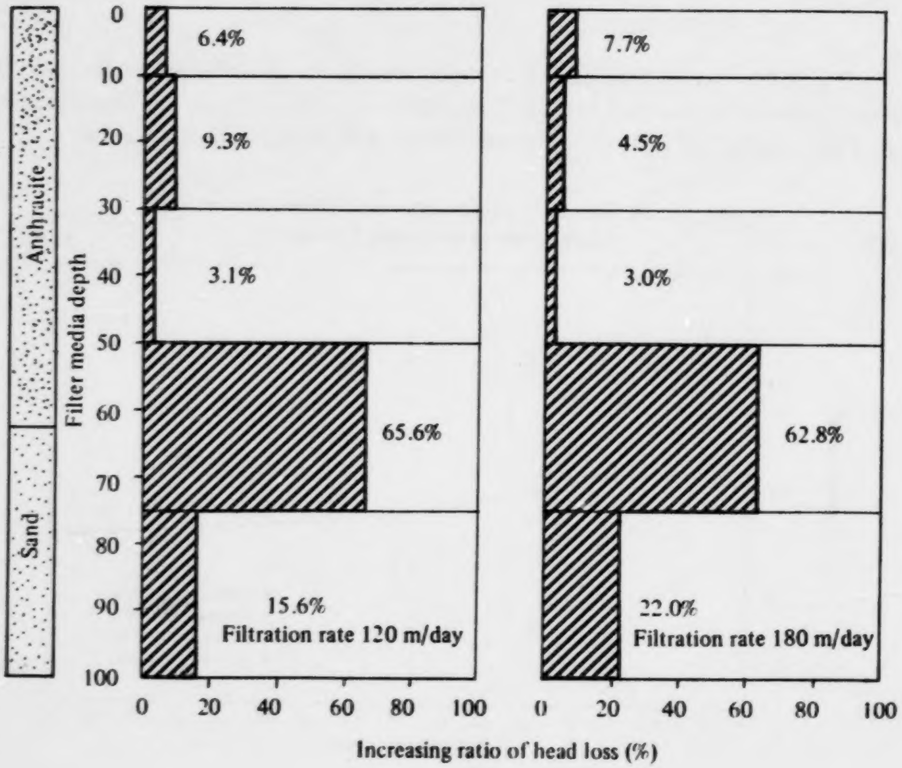
$$[\text{Treated volume } (\text{m}^3/\text{m}^2)] = \frac{[\text{SS loading } (\text{g/m}^2)]}{\left[\begin{array}{c}\text{SS influent}\\ (\text{g/m}^3)\end{array}\right] - \left[\begin{array}{c}\text{SS in effluent}\\ (\text{g/m}^3)\end{array}\right]} \quad \ldots\ldots (5)$$

The treated volume obtained by eq. (5) shows that the less suspended solids in the influent are, the more steeply the treated volume increases below a point of 10 mg/ℓ of suspended solids in the influent. Above the point, the decrease in the treated volume caused by the increase in suspended solids in the influent is not very sharp.

Judging from the data obtained under the conditions of this experiment, it can be concluded that varying filtration is not less efficient than constant filtration in terms of the effluent quality and the SS loading in the case of filtration of alum precipitated effluent.

Fig. 3.2.15 shows the relationship between the filter media depth and the average head loss increase ratio obtained from the head loss distribution in the filter bed during constant filtration. The head loss increase ratio here means the same as defined in **a**.

Fig. 3.2.15 Head Loss ~ Filter Media Depth
(Influent: Effluent from Alum Precipitation of Secondary Effluent)



According to the figure, the head loss increase is almost negligible in the anthracite layer but it occurs in the anthracite-sand border layer, standing at 65.6% and 62.8% respectively for 120 m/day and 180 m/day of the filtration flow rate, or almost 2/3. This means that most floc in the effluent from alum precipitation passes through the anthracite layer and is captured by the sand layer. This tendency is recognized even in the relatively low filtration flow rate of 120 and 180 m/day. Therefore, a conclusion is that anthracite with the effective size of 1.62 used for this experiment is too large for the filtration of the alum precipitated effluent.

3.2.2 SOLID AND ORGANIC REMOVAL BY CARBON CONTACTOR

a. Experimental Precedure

The carbon contractors used for this experiment are the gravity, down flow type contractors with a sectional area of 0.7 m x 1 m = 0.7 m^2. The height of carbon filled is 3 meters. The water level above the carbon is 2 m. The underdrain is the Leopold Block.

There are six contractors, two each place in a series. Influents are filtered secondary effluent for the No. 1 contactor, secondary effluent for the No. 3 contractor and the tertiary alum precipitated effluent for the No. 5 contractor.

The carbon for the primary contractors (No. 1, 3, 5) is X-7000, which is spherical carbon made of coal by Takeda Pharmaceutical Company Ltd,. The carbons for the secondary contractors are Calgon-SGL for the No. 2 contractor, Calgon-CAL for the No. 4 contractor and Takeda's Shirasagi for the No. 6 contractor.

The primary contractors are for the study of suspended solid removal. X-7000 is used for them, because the X-7000 has a relatively large effective size (1.21 mm) and a small uniformity coefficient (1.32) among brands of carbon now available on the market. Table 3.2.2 points to considerable difference existing in the effective size and the uniformity coefficient from one brand to the other. The choice of different brands of carbon for the secondary contractors is for the continuation of carbon regeneration experiments, which have already been conducted four times.

Table 3.2.2 Activated Carbons Used for the Experiment

	Takeda X-7000	Takeda Shirasagi	Calgon SGL	Calgon CAL
Mesh	8 x 32	8 x 30	8 x 30	12 x 40
E.S. (mm)	1.21	0.82	0.72	0.52
U.C.	1.32	2.02	2.06	2.00
Total Surface (m^3/g)	900 ~ 1,200	970	980	1,065
Iodin Number (mg/g)	950 ~ 1,150	950	950	1,000
Methylene Blue Number (mℓ/g)	180 ~ 200	180	170	200
Material	Coal	Coal	Coal	Coal
Shape	Spherical	Crushed	Crushed	Crushed

The filtration flow rate for the experiment is 240 m/day (LV = 10) in constant filtration and 240 m/day on the average in varying filtration. The variation of varying filtration here is identical to that on Fig. 3.2.1. The run is terminated when the total head loss reaches 5 m.

The suspended solids was obtained by measuring turbidity with the surface-scatter type turbidimeter and feeding the turbidity to the equations on the relationship between suspended solids and turbidity. The equations are (cf. Fig. 3.2.2, 3.2.3):

for secondary effluent

$[SS\ (mg/\ell)] = 0.969 \times [turbidity\ (mg/\ell)]$

filter effluent of secondary, effluent contactor effluent of filtered secondary effluent, contactor effluent of secondary effluent.

$[SS\ mg/\ell)] = 0.878 \times [turbidity\ (mg/\ell)]$

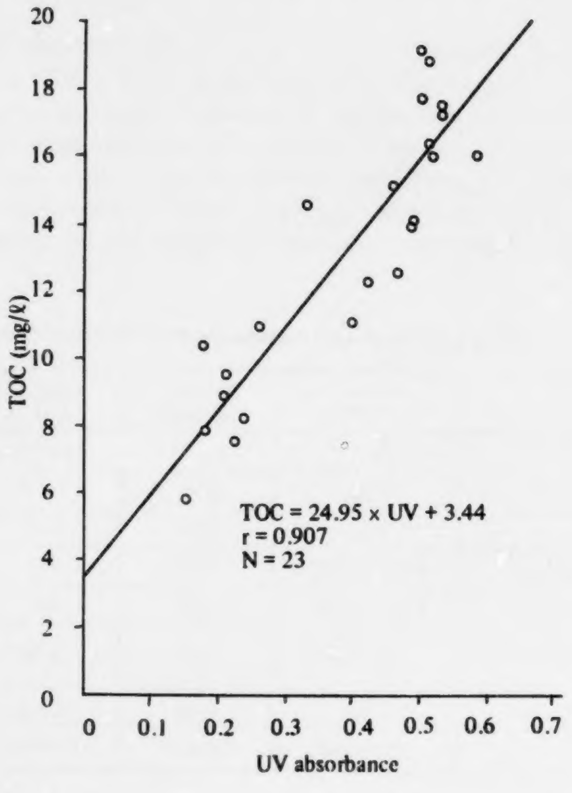
effluent of alum precipitation

$[SS\ (mg/\ell)] = 1.303 \times [turbidity\ (mg/\ell)]$

contactor effluent of alum precipitated effluent

$[SS\ (mg/\ell)] = 0.975 \times [turbidity\ (mg/\ell)]$

Fig. 3.2.16 UV Absorbance ~ TOC



The organics was obtained through the equation on the relations between TOC and UV (254 mm) (cf. Fig. 3.2.16). The UV absorbance needed for the equation was measured by the continuous UV monitor made by Tinsley Co., Ltd,. The equation is:

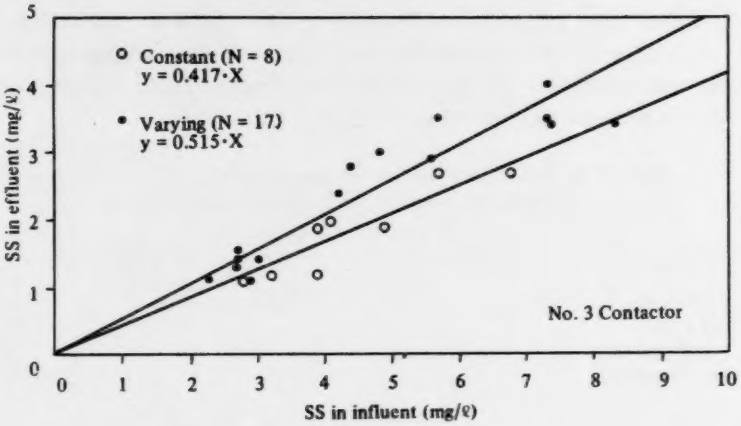
$$\text{TOC (mg/ℓ)} = 25.0 \times \text{UV} + 3.44$$

The UV monitor is designed to measure the absorbance of visible rays too so as to eraze the error resulting from the presence of suspended solids by reducing the visible rays absorbance from that of UV. Except for some special cases, measured values for discussing the result are average values in every case of run.

b. Solid Removal

Fig. 3.2.17 gives the relationship of suspended solids in the influent for the No. 3 contractor (secondary effluent) and in its effluent. The SS removal ratio is independent of suspended solid concentration in the influent by both constant and varying filtration. The ratio is about 58% in constant filtration against 49% in varying filtration. Constant filtration attained a better removal by about 10% than varying filtration. However, it is a matter for future problem whether this difference is of any significance, because the two methods were tested at different times – constant filtration in November and varying filtration in December.

Fig. 3.2.17 SS Removal by Carbon Contactor (Influent : Secondary Effluent)



The comparison between SS in the effluent from the No. 1 contractor (influent – filtered secondary effluent) and that from the No. 3 contractor shows that the No. 3 contractor is a little less efficient than the No. 1 contractor. But the difference was only 9%. The average SS in the effluent from the No. 1 contractor was 2.0 mg/ℓ against 2.2 mg/ℓ in the effluent from the No. 3 contractor (Fig. 3.2.18). Therefore, it seems that as far as suspended solid removal no major problem will arise from omitting filter.

Fig. 3.2.18 Comparison of SS between No. 1 Contactor Effluent and No. 3 Contactor Effluent

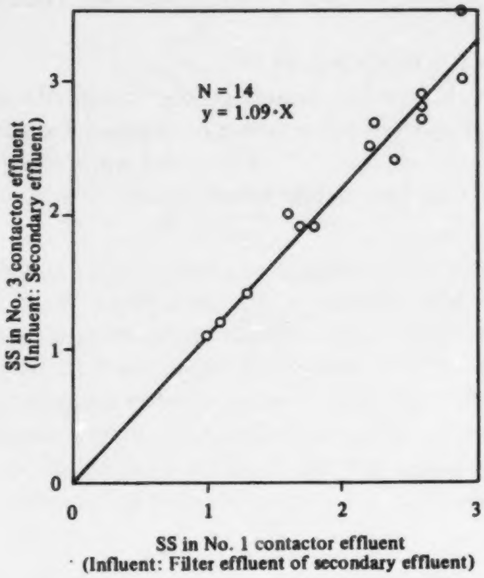
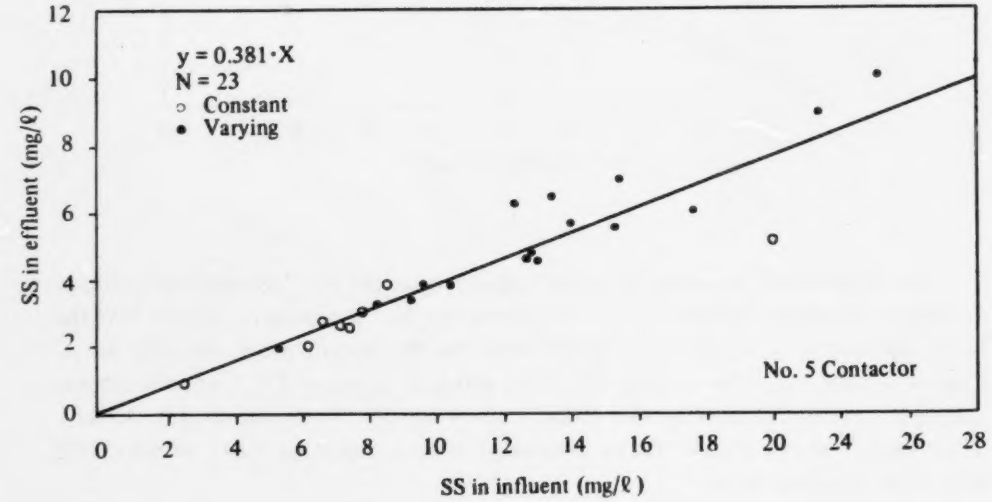


Fig. 3.2.19 shows the relationship of suspended solids in the influent of the No. 5 contractor (tertiary alum precipitated effluent) and in its effluent. When the effluent from alum precipitation was treated by the contractor, no difference was found with regard to the SS removal between constant and varying filtration with about 62% in both methods. In the two months of experiment, the average SS was 11.8 mg/ℓ for the influent against 4.5 mg/ℓ for the effluent.

Fig. 3.2.19 SS Removal by Carbon Contactor (Influent : Effluent from Alum Precipitation of Secondary Effluent)



In most runs on the No. 3 contractor, washing was started before the total head loss reaching five meters. It was only once that the SS loading was measured at the point of the total head loss reaching five meters. However, it is possible to estimate the SS loading of constant filtration through measured values on the rate of the head loss against the SS loading on Fig. 3.2.21. And the value estimated is an average 3,700 g/m² (2,100 ~ 4,600 g/m²) against average SS in the influent being 4.8 mg/ℓ.

Fig. 3.2.20 SS Loading of Carbon Contactor (Influent : Effluent from Alum Precipitation of Secondary Effluent)

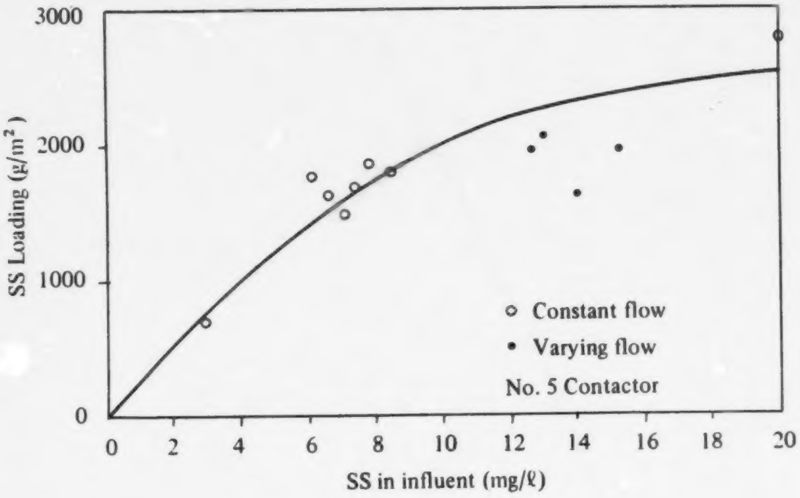
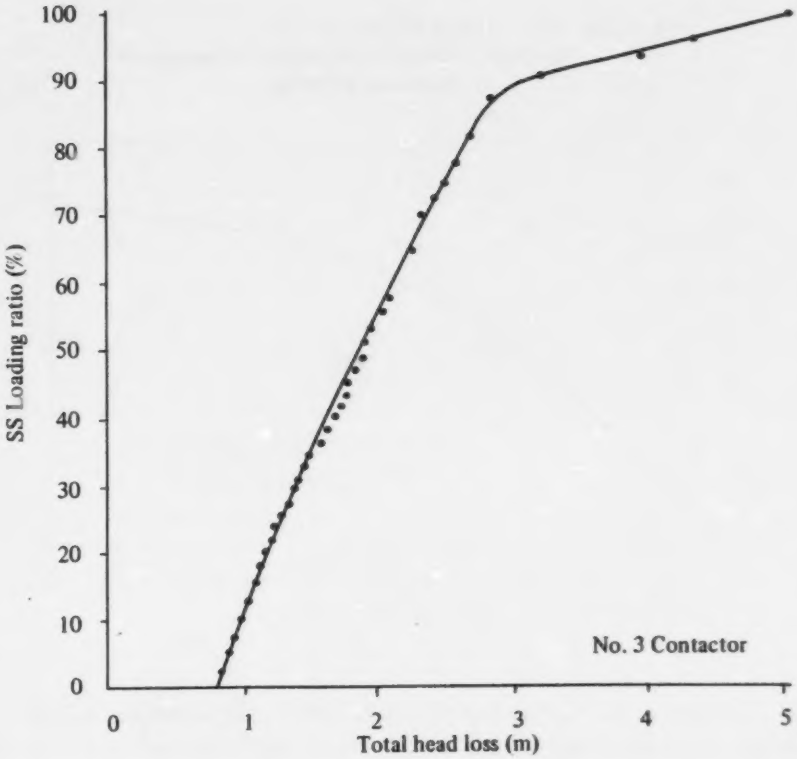


Fig. 3.2.20 shows the relationship of the SS loading to suspended solids in the influent at the total head loss of 5 m in the No. 5 contractor (influent – tertiary alum precipitated effluent). The SS loading rises with the increase of suspended solids in the influent in constant filtration. The loading of varying filtration is too limited to give any definite conclusion. But the value of varying filtration is estimated to be smaller than in constant filtration.

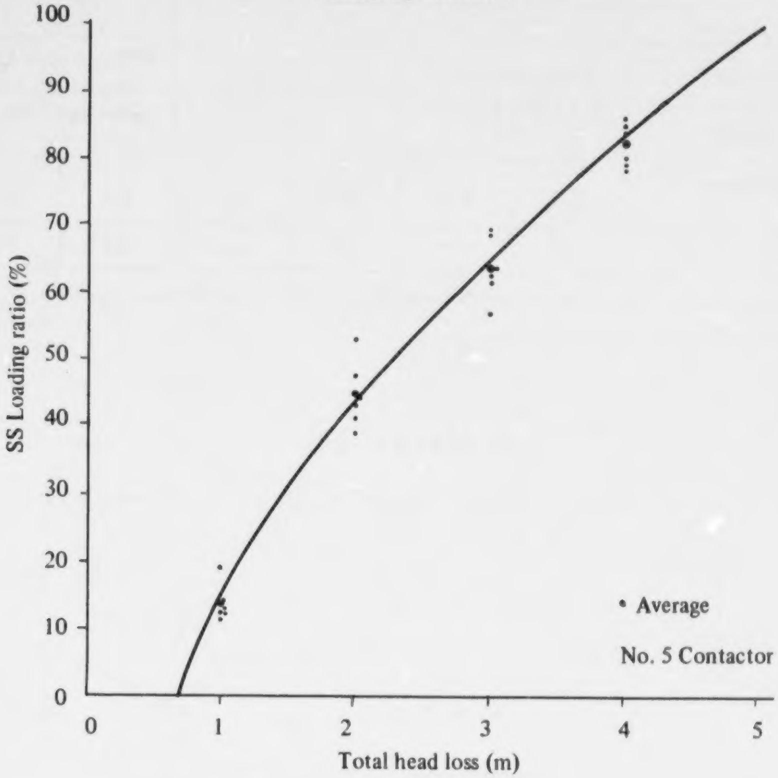
Fig. 3.2.21 and 3.2.22 show the SS loading ratio under lower total head loss than 5 m against 5 m. Fig. 3.2.21 gives results of the No. 3 contractor, while Fig. 3.2.22 is those of the No. 5 contractor.

Fig. 3.2.21 Total Head Loss ~ SS Loading Ratio
(Influent : Secondary Effluent)



In the case of the No. 3 contractor using the secondary effluent as influent, the SS loading goes up almost linearlly with the increase of the total head loss up to 3 m. But when the total head loss exceeds 3 m, the SS loading increasing ratio for every unit total head loss drops sharply. About 90% of the SS loading at 5 m of the total head loss is achieved by 3 m. This may probably mean that the filtration goes into the complete surface filtration at the total head loss of about 3 m because of accumurated suspended solids in the surface area of carbon. But it must be repeated that there was only one case of the total head loss in the No. 3 contractor reaching to 5 m. Fig. 3.2.21 shows the results of this single case. More study is needed to know exactly about whether the downturn in the SS loading occurs at the total head loss of 3 m or whether 90% of the SS loading at the head loss of 5 m is accomplished already at the point of 3 m.

Fig. 3.2.22 Total Head Loss ~ SS Loading Ratio (Influent : Effluent from Alum Precipitation of Secondary Effluent)



Any development of this kind did not happen in the No. 5 contractor for which the effluent from alum precipitation was used as influent. The SS loading of the contractor increased in keeping pace with the increase of the total head loss to 5 m, indicating the effective use of the total head loss. This also means that the No. 5 contractor has more of the nature of in-depth filtration than the No. 3 contractor.

c. Organic Removal

Table 3.2.3 shows average of TOC removal in the two months of experiment. TOC in the effluent from the No. 1 contractor is 9.8 mg/ℓ against 10.2 mg/ℓ in the No. 3 contactor effluent. The difference was 0.4 mg/ℓ, or 4%. In view of this, it could be concluded that there is little need of filter for TOC removal at present.

Fig. 3.2.23 shows the relationship between C/Co and BV in the contractors No. 1 and 2. Fig. 3.2.24 gives the ratio of TOC in the effluent from the No. 1 contractor to that from the No. 3 contractor. Judging from Fig. 3.2.23, carbon appears to be breaking, but this must be more precisely examined by future results. And at least up to Bed Volume (BV) of 5,000, there is no visible change in the ratio of TOC in the effluent from the No. 1 contractor to that from the No. 3 contractor.

Table 3.2.3 TOC Removal

Contactor No.*	1	2	3	4	5	6
Number of Data	20		20		15	
Influent to the Primary Contactor and its TOC (mg/ℓ)	Filter Effluent of Secondary Effluent 14.1		Secondary Effluent 14.5		Effluent from Alum Precipitation of Secondary Effluent 15.0	
TOC of Effluent (mg/ℓ)	9.8	4.6	10.2	5.1	8.7	4.6
TOC Removal	30	67	30	65	42	69

* Contactor Nos. 1, 3 and 5 are the primary contactors, and 2, 4 and 6 the secondary.

** To the influent to the primary contactor.

Fig. 3.2.23 BV ~ C/Co

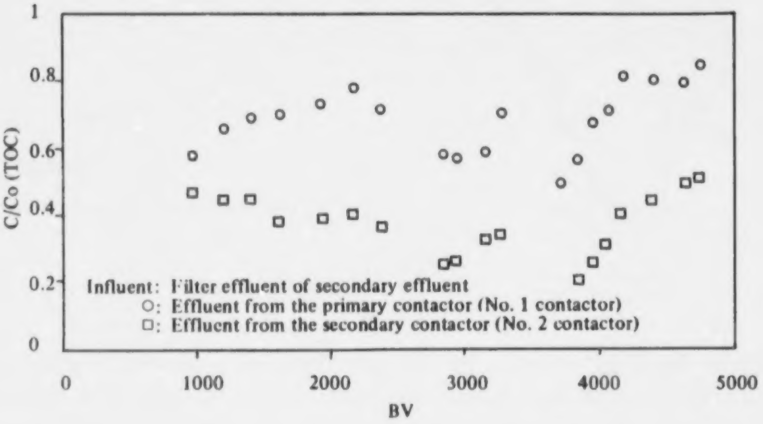
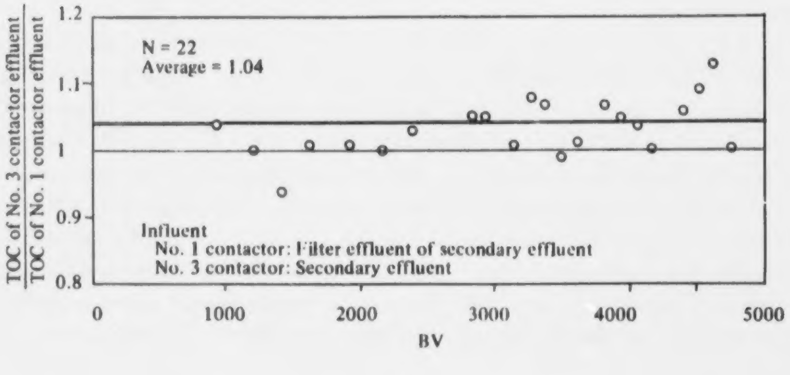


Fig. 3.2.24 Comparison of TOC between No. 1 Contactor Effluent and No. 3 Contactor Effluent



3.2.3 SUMMARY

Table 3.2.4 gives the summary of the removal of suspended solids by the filter and the carbon contractor in terms of the SS removal ratio, the SS loading and the treated volume under specific operating conditions.

Followings are the findings of this study:

Table 3.2.4 Summary

	Influent	Filtration Method	Filtration Rate (m/day)	Suspended Solid Removal										Filtered Volume (m^3/m^2)				
				Removal (%)					SS Loading (g/m^2)					SS in Influent				
				SS in Influent (mg/ℓ)					SS in Influent (mg/ℓ)									
				2.5	5	7.5	10	20	2.5	5	7.5	10	20	2.5	5	7.5	10	20
Filter	Secondary Effluent	Constant	180	61	61	61	–	–	–	1,700	2,300	–	–	–	560	500	–	–
				* y = 0.387·x + 0.02														
			300	61	61	61	–	–	–	3,000	3,500	–	–	–	980	770	–	–
				* y = 0.387·x + 0.02														
			420	61	61	61	–	–	–	3,000	3,500	–	–	–	980	770	–	–
				* y = 0.387·x + 0.02														
			500	–	42	48	52	57	–	1,700	2,300	–	–	–	810	640	–	–
				* y = 0.381·x + 1.01														
		Varying	300	–	42	48	52	57	–	3,100	2,800	–	–	–	980	770	–	–
				* y = 0.381·x + 1.01														
			420	–	42	48	52	57	–	2,100	2,800	–	–	–	980	770	–	–
				* y = 0.381·x + 1.01														
	Tertiary Alum Precipitated Effluent	Constant	120	62	55	53	52	50	490	650	820	980	1,600	310	240	210	190	160
				* y = 0.521·x – 0.36					y = 66·x + 320									
			180	62	55	53	52	50	490	650	820	980	1,600	310	240	210	190	160
				* y = 0.521·x – 0.36					** y = 66·x + 320									
			300	62	55	53	52	50	490	650	820	980	1,600	310	240	210	190	160
				* y = 0.521·x – 0.36					** y = 66·x + 320									
		Varying	180	62	55	53	52	50	490	650	820	980	1,600	310	240	210	190	160
				* y = 0.521·x – 0.36					** y = 66·x + 320									
Carbon Contactor	Filtered Secondary Effluent	Constant	240	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
		Varying		–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	Secondary Effluent	Constant		58	58	58	–	–	–	–	–	–	–	–	–	–	–	–
				* y = 0.417·x														
		Varying		49	49	49	–	–	–	–	–	–	–	–	–	–	–	–
				* y = 0.515·x														
	Tertiary Alum Precipitated Effluent	Constant		62	62	62	62	62	700	1,200	1,700	2,000	–	450	390	370	320	–
				* y = 0.381·x														
		Varying		62	62	62	62	62	–	–	–	–	–	–	–	–	–	–
				* y = 0.381·x														

* y: SS in Effluent (mg/ℓ), x: SS in Influent

** y: SS Loading, x: SS in Influent

a. Varying and Constant Filtration

1) When the secondary effluent is used as influent for filter, the comparison between varying and constant filtration at the filtration flow rate of 300, 420 m/day leads to the following conclusions: The effluent by varying filtration contain more of suspended solids by about 1 mg/ℓ than that by constant filtration. The SS loading by varying filtration becames lower by 20 ~ 30%. The lower efficiency of varying filtration is also observed as to the SS removal ratio in carbon contractors using the secondary effluent as influent by 10% drop.

2) When the effluent from alum precipitation is used as the influent for filter or carbon contractor, there is little difference of the SS removal and the SS loading between varying and constant filtration in this study. In carbon contractors, varying filtration is a little less efficient in terms of the SS loading.

3) As seen in varying filtration experiments in the filter using the secondary effluent as influent, time when the total head loss reach to 3 m is limited to the closing period of influent flow increasing or to the time of high influent flow. About 50% of the runs it comes within one hour in the closing period of effluent volume increase.

b. Filtration of the Secondary Effluent by Filter (Constant Filtration)

1) In the filter media composition used for this experiment, the optimum filtration flow rates in terms of the effluent quality, and the SS loading are 300 ~ 420 m/day. This is understood by the fact that the head loss distribution in the filter bed came very close to that of in-depth filtration.

2) When the rates are 300, 420 m/day, and SS in the influent is 5 mg/ℓ, the SS removal is 61% and the SS loading is 3,000 g/m^2.

3) Filtration at the rate of 180 m/day seems to be the surface filtration. The rate, when compared to the higher rates of 300, 420 m/day, provides no difference as far as the SS removal ratio. But tne SS loading of this rate is about 60% of that of 300, 420 m/day.

4) The removal of suspended solids by the anthracite layer cannot be expected at the rate of 500 m/day. Most of the SS capture takes place in the sand layer. At this rate the SS in the effluent is by about 1 mg/ℓ greater than at the rates of 300, 420 m/day. The SS loading is about 60% of the value for the rates of 300, 420 m/day.

c. Filtration of Alum Precipitated Effluent by Filter (Constant Filtration)

1) In the rates of 120 ~ 300 m/day, there is no difference in the SS removal and the SS loading. When the SS in the influent is 5 mg/ℓ, the SS removal rate is 55% and the SS loading 650 g/m^2.

2) The SS removal by anthracite could not be expected. Most of the SS capture takes place in the sand layer. This means that the anthracite in this bed, having the effective size of 1.62, is large for the treatment of the effluent from alum precipitation.

d. Solid Removal by Carbon Contractor

1) When treated by the carbon contractors, the filter effluent of the secondary effluent and the secondary effluent itself make no great difference in their effluent quality. The quality difference is only 9% for suspended solids and 4% for TOC up to BV 5,000.

2) When compared to filter filtration of the secondary effluent, there is no great difference in the SS removal efficiency. The carbon contractor also is not inferior in terms of the SS loading.

3) When the effluent from alum precipitation with SS of 5 mg/ℓ is used as influent, the SS removal ratio is 55% and the SS loading is 1,200 g/m^2 (the total head loss-5 m). When the total head loss is 3 m, the SS loading will be 65% of the

value at 5 m of the total head loss, or 780 g/m².

4) When compared to filter filtration by using the effluent from alum precipitation as influent, the carbon contractor is superior both in the SS removal ratio and the SS loading.

e. Filtration of Alum Precipitated Effluent and Secondary Effluent

1) In the case of the filter filtration at the flow rates of 180, 300 m/day, the SS removal and the SS loading is better in secondary effluent filtration. The SS loading of the effluent from alum precipitation was about 30% of that of the secondary effluent.

2) In the case of the carbon contactor, the SS removal rate of the effluent from alum precipitation was better by 3% but its SS loading was smaller than that of the secondary effluent.

f. Future Studies to be Conducted

1) Varying filtration experiments by changing the filtration rate gradient or by changing the ratio of the maximum flow rate to the minimum flow rate.

2) Dual or multi media filtration of the effluent from alum precipitation by using anthracite with the effective size smaller than 1.62 mm.

3) To compare constant and varying filtration by using the same filter to verify the results above. The experiment is now in progress.

4) To examine the carbon contractor filtration including the aspect of carbon regeneration. Up to now it is known that the treatment of sewage including suspended solids by the carbon contractor produces no major problem with regard to SS and TOC in the effluent. Also needed is to know the performance of the carbon contractor in treating the influent of high SS. The average SS in the secondary effluent used so far is low at 4.5 mg/ℓ.

5) To check whether there is a drop in the absorption speed by using coarse carbon, which have proved to be efficient for the removal of suspended solids.

CHAPTER 4. EXPERIMENTAL STUDY ON REGENERATION OF GRANULAR ACTIVATED CARBON

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4. EXPERIMENTAL STUDY ON REGENERATION OF GRANULAR ACTIVATED CARBON

INTRODUCTION

In wastewater treatment, particularly in advanced wastewater treatment, it increases such a case that granular activated carbon is used as a means of removal of soluble organic substances.

The unit cost of granular activated carbon is extremely expensive because a good quality of bituminous coal as its raw material is difficult to obtain in Japan and the product yield from such raw material is very small. These are the principal defect of granular activated carbon when we used it for wastewater treatment. Consequently if spent carbon can simply be regenerated at the inexpensive cost, the utilization of granular activated carbon to wastewater treatment will be increased.

The regeneration of granular activated carbon is normally carried out by the manufacturers, but in the case of wastewater treatment which requires activated carbon is quantity equivalent to approximate 1/50 and (approximate 1/100 by weight) more of wastewater to be treated per day, more regeneration on site is required as the scale of wastewater treatment becomes larger.

The following are the results of our experiments we have conducted four items on the activated carbon used in advanced wastewater treatment of the Kyoto pilot plant by a multiple hearth regeneration furnace.

4.1 METHOD IN THE EXPERIMENTS

4.1.1 GRANULAR ACTIVATED CARBON USED FOR EXPERIMENTS

The brands and particle sizes of activated carbon used for our experiments are shown in the Table 4.1 and these types of activated carbon have been used for the adsorption of rapidly filtered secondary effluent in the Kyoto pilot plant. In our adsorption experiments, no fresh carbon has been added since we intended to observe the changing characteristics of the original carbon with repeat of regeneration. The approximate figures of adsorbed of COD_{Mn} obtained from the result of adsorption experiment is also shown in Table 4.1. The COD_{Mn} removal in treatment just before regeneration was approximately 30% in each case.

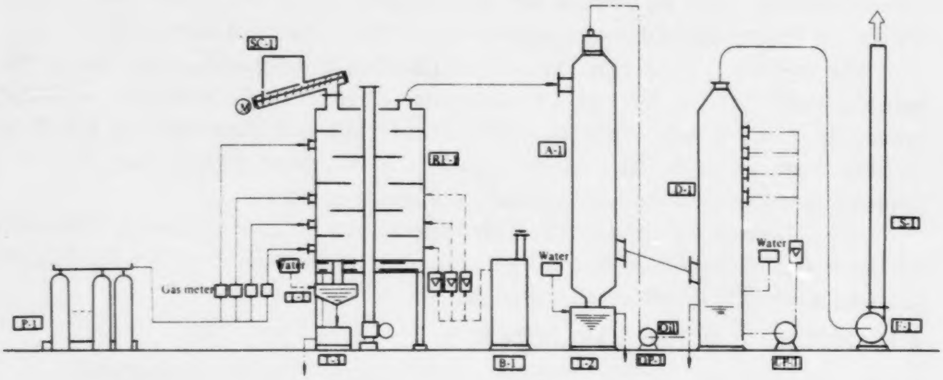
4.1.2 FACILITIES USED IN EXPERIMENTS

The facilities used for our experiments comprised a regeneration furnace, an equipment for feeding and collecting carbon, an equipment for treating exhaust gas and a steam generator. A general flow is as shown in Fig. 4.1. The regeneration furnace is a vertical type six stage hearth furnace having the inside diameter of 750 mm and 1,500 mm of the height which is equipped with eight gas burners. The arms of the furnace can be rotated within the range of 0.23 to 2.3 rpm by a 0.4 kW driving motor.

Table 4.1 Carbon Used in Experiment and Adsorbed COD_{Mn}

Number of Reactor	1	2	3	4	5	6
Name and Size						
Name	Calgon, SGL		Calgon, CAL		Takeda, Shirasagi	
Particle Size (Mesh)	8 x 30		12 x 40		8 x 30	
Adsorbed COD_{Mn} (COD_{Mn} kg/kg A.C.)						
1st	0.118	0.074	0.160	0.088	0.121	0.085
2nd	0.150	0.067	0.170	0.080	0.155	0.057
3rd	0.132	0.047	0.160	0.061	0.086	0.048
4th	0.165	0.067	0.202	0.079	0.125	0.072
Mean	0.141	0.064	0.189	0.077	0.122	0.066

Fig. 4.1 Flow Diagram of the Regeneration System



P-1	Fuel gas	A-1	Exhaust gas furnace	T-1	Quench tank	CP-1	Recycle pump
SC-1	Screw conveyer	D-1	Scrubber	T-2	Seal tank		
RF-1	Regeneration furnace	S-1	Chimney	T-3	Dewatering tank		
OP-1	Oil pump	B-1	Boiler	F-1	Exhaust gas fan		

4.1.3 ITEMS MEASURED

The items indicating the characteristics of granular activated carbon that measured in this experiment are; physical characteristics (mean particle size, hardness number, ignition residue, specific surface area, mean micropore size, volume of micropore, micropore distribution), general adsorption characteristics (methylen blue decolorizing capacity, iodine adsorption capacity, molasses decolorizing capacity, phenole value, ABS value), adsorption capacity of organic substances in water and weight ratio of adsorbed matter to fresh carbon.* Measuring has been imple-

---

*: Weight ratio of adsorbed matter to fresh carbon can be obtained by the following; a 100 kg of dried sample of spent carbon will be heated for 90 minuts in an electrical furnace, the temperature of which was controlled to 900°C in N_2 gas flow, and the decreasing of weight are measured, then the weight ratio will be obtained according to the following formula;

$$\text{Weight Ratio of Adsorbed Matter to Fresh Carbon} = \frac{\text{Decreased Weight of Carbon (g)}}{\text{Weight of Recovered Carbon (g)}}$$

mented in accordance with JIS standards and Japan Water Works Association (AWWA) Standards as far as practicable, but the details of the method have been omitted because it was described in the previous report.

4.1.4 REGENERATING CONDITIONS

Regenerating efficiency of granular activated carbon in the regeneration furnace will be governed by the following factors;

(1) Temperature and its distribution in furnace
(2) Detention time of activated carbon in furnace
(3) Atmosphere in furnace
(4) Feed rate of steam for reactivation and its feeding position
(5) Loading rate of carbon

In our experiments, the furnace temperature has been kept in 930 to 940°C at a maximum. Each stage of 4th, 5th and 6th hearth for activating has been so set as to be able to maintain nearly the same temperature. The detention time of activated carbon in the furnace has been set for 30 min. regardless of the type and loading quantity of carbon. Atmosphere in the furnace has been so controlled that O_2 for each stage should be kept below 0.1% to avoid the burning-off of carbon due to the entrance of air.

The activating temperature has been changed based on the type of activated carbon and apparent density which show the result for any change of loading quantity and can be measured easily. One third of the steam has been fed to the stage 4 and the remainder to the stage 6.

The loading rate of activated carbon has been set at from 11 to 15 kg/m^2 of furnace floor/hr on the basis of dry spent carbon by weight.

Total feeding weight or volume and loading quantity/hr in the regeneration experiments carried out four times are as shown in Table 4.2. The activated carbon

Table 4.2 Feeding Weight and Feeding Rate of Each Carbon in the Experiment

Items	Regeneration \ Reactor	1	2	3	4	5	6
Weight of Feeded Activated Carbon (DSC kg)	1st	836.0	836.0	696.7	727.5	809.0	835.9
	2nd	784.9	752.3	681.5	645.6	798.0	791.7
	3rd	670.9	601.9	577.6	567.7	735.8	693.3
	4th	618.5	546.5	596.2	550.5	733.3	686.3
	Mean	727.6	684.2	563.0	622.8	769.0	751.8
Weight of Feeded Activated Carbon per Hour (kg DSC/Hr)	1st	39.5	34.3	33.6	32.1	35.6	34.0
	2nd	34.2	32.4	36.9	31.0	38.2	36.8
	3rd	33.5	31.7	28.1	28.8	35.0	32.8
	4th	34.4	33.0	32.5	30.2	36.1	33.9
	Mean	35.4	32.9	32.8	30.5	36.2	34.4

withdrawn from the reactor has been put into the hemp sack and dewatered when the specific gravity was stabilized in wet condition. The activated carbon has been loaded by weighing each sack of carbon.

In adsorption experiments at the pilot plant, each of the carbon was divided and packed into two vessels. And the vessels were operated in series. So, this regeneration experiment has been implemented by two vessels in a similar manner. The first experiment has been carried out in December, 1974, the second experiment in August, 1975, the third experiment in February, 1976 and the fourth experiment in August, 1976 respectively.

4.2 RESULTS OF EXPERIMENTS

4.2.1 RECOVERY RATE WITH REGENERATION

The recovery rate of activated carbon with regeneration can be expressed both gravimetrically and volumetrically. It is generally said that the reliability of the former is greater than that of the latter although the former has problems that we cannot grasp any change of particle size arising from mechanical and thermal losses and that we cannot distinguish the influence of accumulation of inorganic substances.

Gravimetric recovery rate (Rw) can be obtained from dry weight of spent carbon (Ws), weight ratio of adsorbed matter to fresh carbon (a) and dry weight of regenerated carbon (Wr).

$$Rw = \frac{Wr\,(1 + a)}{Ws}$$

The volumetric recovery rate can further be obtained 1) from the height of packed carbon and cross sectional area of the reactor and 2) from volumetric figures calculated by dividing carbon weight with apparent density.

Each recovery rate obtained from this experiment is as indicated in Table 4.3. Each recovery rate varies by the type of carbon and by the frequency of use, but was approximately 85% and above.

The fluctuation of height of packed carbon in reactor and apparent density (measured by a 100 mℓ measuring cylinder) with repeat of regeneration is as shown in Fig. 4.2. In Fig. 4.2 and other diagrams, F, S and R on horizontal axis denote fresh carbon, spent carbon and regenerated carbon, respectively. The numbers show the frequency of adsorption and regeneration. From Fig. 4.2, it is noted that apparent density relatively reduces as the carbon is regenerated repeatedly and that the height of packed carbon lowers linearly. In accordance with that, carbon weight of each reactor is reduced to approximate 50% of the original carbon weight only by four times regeneration although it include a loss in back washing.

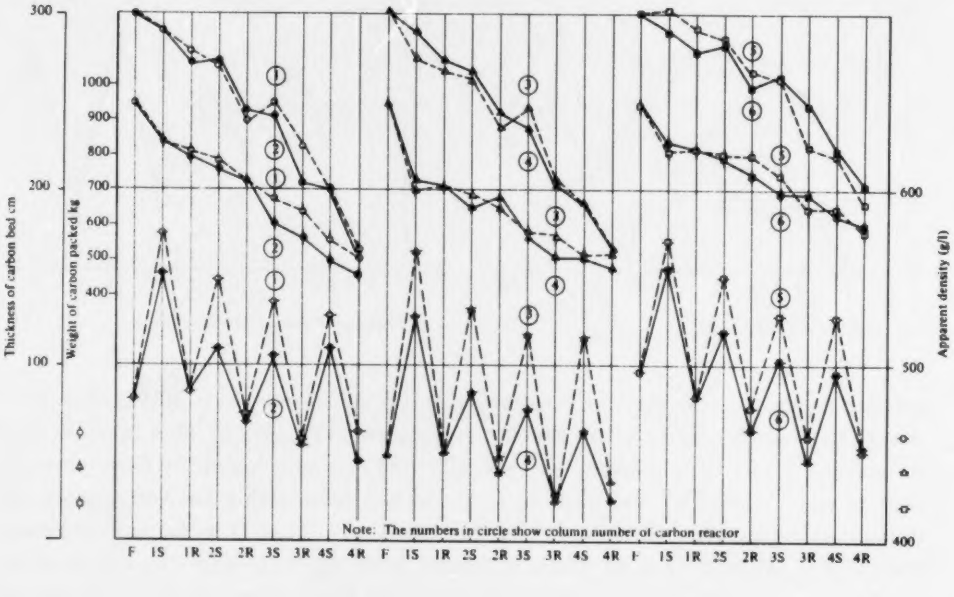
4.2.2 PHYSICAL CHARACTERISTICS CHANGE OF GRANULAR ACTIVATED CARBON

Out of physical characteristics of the activated carbon, Fig. 4.3 shows the change occurring from the repeated regeneration on mean particle size, hardness, ignition residue, specific surface area and mean micropore size of micropore 0 to 300Å. From these figures, it can be seen that by the repeat of regeneration, mean particle size and hardness have not been changed very much, but ignition residue,

Table 4.3 Recovery Rates in Regeneration Test (%)

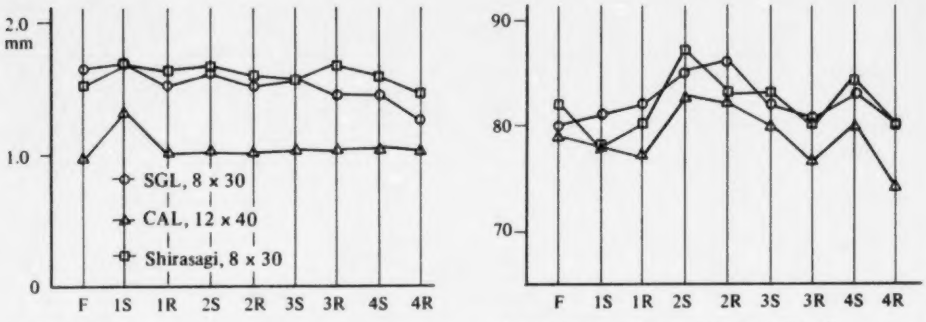
Items	Regeneration \ Reactor	1	2	3	4	5	6
Gravimetric Recovery Rate	1st	96.2	94.0	102.3	98.1	100.2	97.0
	2nd	91.9	96.8	95.1	105.1	99.6	93.8
	3rd	94.7	93.0	98.5	89.7	87.7	98.7
	4th	90.8	91.9	99.8	94.1	89.6	95.1
	Mean	93.4	93.9	98.9	96.8	94.3	96.2
Volumetric Recovery Rate 1	1st	95.6	94.1	97.4	94.1	95.8	96.1
	2nd	88.0	89.6	90.0	91.1	93.4	91.8
	3rd	90.2	84.7	83.2	86.3	84.8	93.4
	4th	83.8	80.0	86.1	87.1	87.4	90.6
	Mean	89.4	87.1	89.2	89.7	90.4	93.0
Volumetric Recovery Rate 2	1st	99.3	96.1	107.7	98.2	99.8	98.5
	2nd	92.7	94.0	96.7	100.0	98.5	92.2
	3rd	100.6	95.7	105.2	90.0	90.8	101.4
	4th	93.0	95.9	103.0	94.5	92.2	95.2
	Mean	96.4	95.4	103.2	95.7	95.3	96.8

Fig. 4.2 Change of Thickness, Weight and Apparent Density of Carbons with Repeat of Regeneration



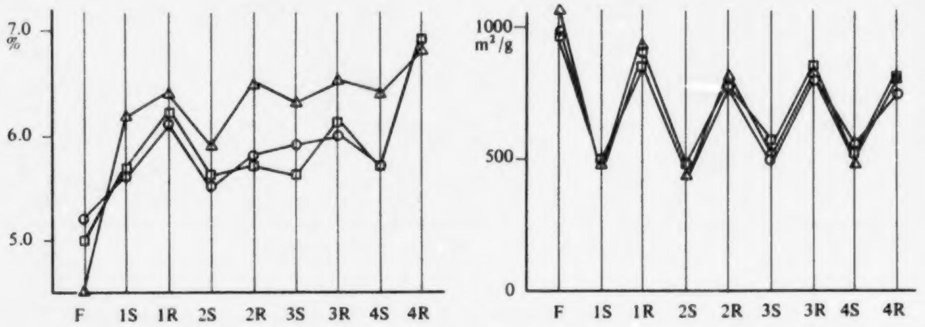
Note: The numbers in circle show column number of carbon reactor

Fig. 4.3 Physical Characteristic Change of Granular Activated Carbons with Repeat of Regeneration



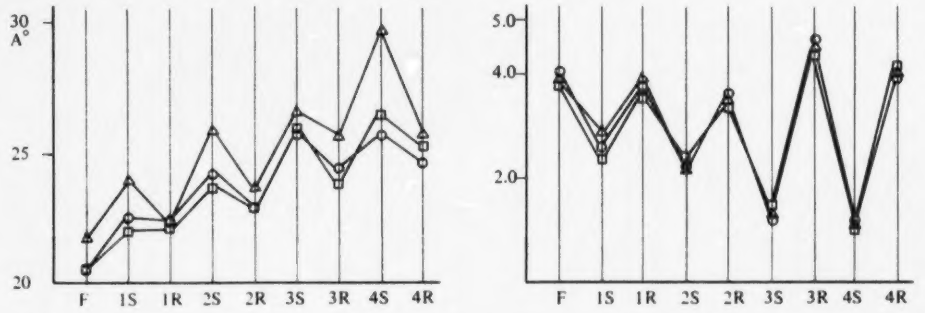
(a) Mean particle size

(b) Hardness value (Remaining rate in shieve)



(c) Ignition residue

(d) Specific surface area

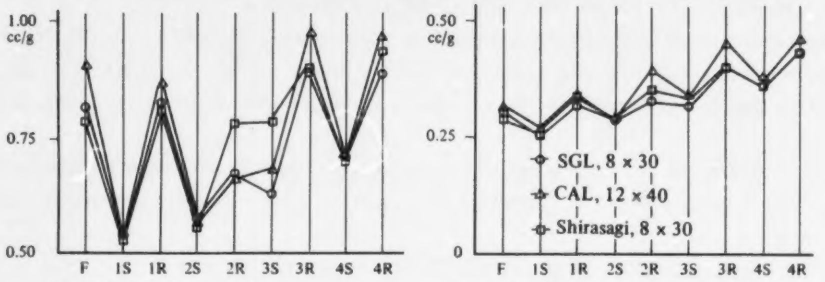


(e) Mean pore size of 0~300Å

(f) Converse of phenol value (x 100)

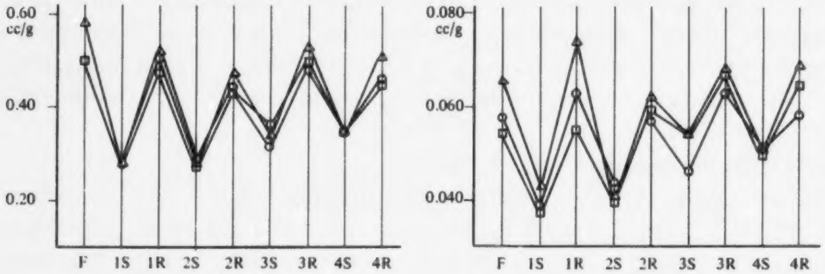
specific surface area, mean micropore size of micropore 0 to 300Å have been changed strikingly. Particularly specific surface area (Fig. 4.3 (d)) has been reduced to nearly a 50% of the original area just after the regeneration at the time when the carbon was spent. This explains well the mutual relationship between adsorption status and regenerating efficiency. Further, ignition residue (Fig. 4.3 (c)) and mean pore size of micropore 0 to 300Å increased little by little with the repeat of regeneration. The reduction of these values shows the progressing of the deterioration of the activated carbon considering the reduction of adsorption capacity as will be described later on. From the investigation of another physical characteristics of micropore volume, the volume of each micropore size fraction varies with the repeat of regeneration as shown in Fig. 4.4. The total volume of micropore (Fig. 4.4 (a)) of the spent carbon showed a reduction of its value to 60 ~ 70% of fresh carbon or regenerated carbon. The difference in the micropore volume from 300Å to 15 μ (Fig. 4.4 (b)) between fresh carbon and spent carbon is relatively small, but the volume increased gradually with repeat of regeneration. On the contrary the micropore volume (Fig. 4.4 (f)) up to 12Å varied considerably and the value reduced with repeat of regeneration. The difference in the value between spent and regenerated carbon is greater on the volume of micropore size fraction, but the value has not been noticeably increased nor decreased very much with repeat of use and regeneration.

Fig. 4.4 Micropore Characteristic Change of Granular Activated Carbons with Repeat of Regeneration



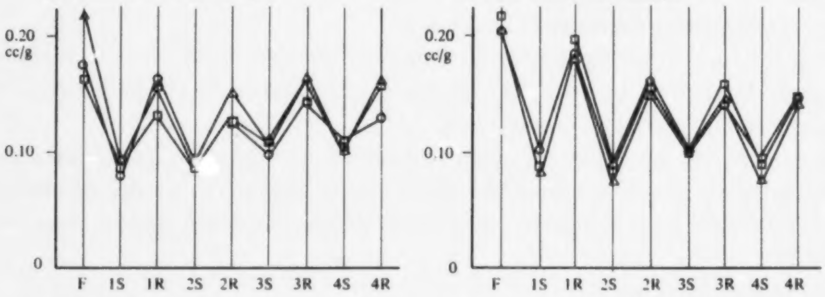
(a) Total volume of 0 ~ 15μ micropore

(b) Volume of 300Å ~ 15μ micropore



(c) Volume of 0 ~ 300Å micropore

(d) Volume of 30 ~ 60Å micropore



(e) Volume of 12 ~ 30Å micropore

(f) Volume of 0 ~ 12Å micropore

From the above, we can consider that with repeat of use and regeneration, the total volume of micropore has not been changed very much, but micropore volume of 12Å and less has been reduced and in its stead micropore from 300Å to 15 μ has been increased. This indicates a deterioration of activated carbon.

Furthermore, in regard to micropore, the distribution has been measured in each regenerating experiment, but here as an example we have shown a change of micropore distribution after the second and the fourth regeneration in comparison with that of fresh carbon in Fig. 4.5.

From Fig. 4.5, it is noticeably that micropore reduced and micropore increased.

4.2.3 CHANGE OF GENERAL CHARACTERISTICS

As an index of adsorption capacity of the activated carbon, methylene blue decolorizing capacity, phenole value and ABS value are generally used. The change of such an index with repeat of regeneration in this experiment is as shown in Fig. 4.6.

Generally speaking each item has been much changed with repeat of regeneration and the value of spent carbon, fresh carbon, and regenerated carbon varied widely in each case.

The following is the study on the characteristics of each index;

Methylene Blue Decolorizing Capacity (Fig. 4.6 (a))

The fluctuation of the capacity has sometimes been reduced to 40 to 50% and was gradually reduced with repeat of regeneration. This index is proportionate to the specific surface area of micropore of 15Å and more, and is so much alike the fluctuation of the specific surface area in (Fig. 4.4 (d)) or micropore volume of 0 to 12Å in (Fig. 4.5 (f)).

Iodine Adsorption Capacity (Fig. 4.6 (b))

The fluctuation of the capacity is such that the capacity of fresh carbon has sometimes been reduced to 20 to 30%, but the value decreased with repeat of regeneration as shown in Fig. 4.5 (f) and Fig. 4.6 (a). This index is proportionate to the specific surface area of micropore of 10Å and more.

Molasses Decolorizing Capacity (Fig. 4.6 (c))

The fluctuation of capacity is such that the capacity of fresh carbon has been reduced to 25 to 50%, but the value shows a reverse trend of the above two items and increases with repeat of regeneration.

It is generally said that this index is proportionate to the specific surface area of micropore of 28Å and more, but judging from the trend on the diagram, the index is rather alike the change of micropore volume from 300Å to 15 μ as shown in Fig. 4.4 (b).

Phenole Value (Fig. 4.6 (d))

This index indicates the required amount of activated carbon to adsorb the unit amount of phenole. Consequently, the smaller this value, the greater the adsorption capacity. From Fig. 4.6 (d), the fluctuation of the value is greater, but even if the capacity considerably reduces by the spending, the original capacity can be restored by regeneration. The capacity will not be dropped much even after the repeated regeneration.

Fig. 4.5 Micropore Distribution Change of Granular Activated Carbons with Repeat of Regeneration

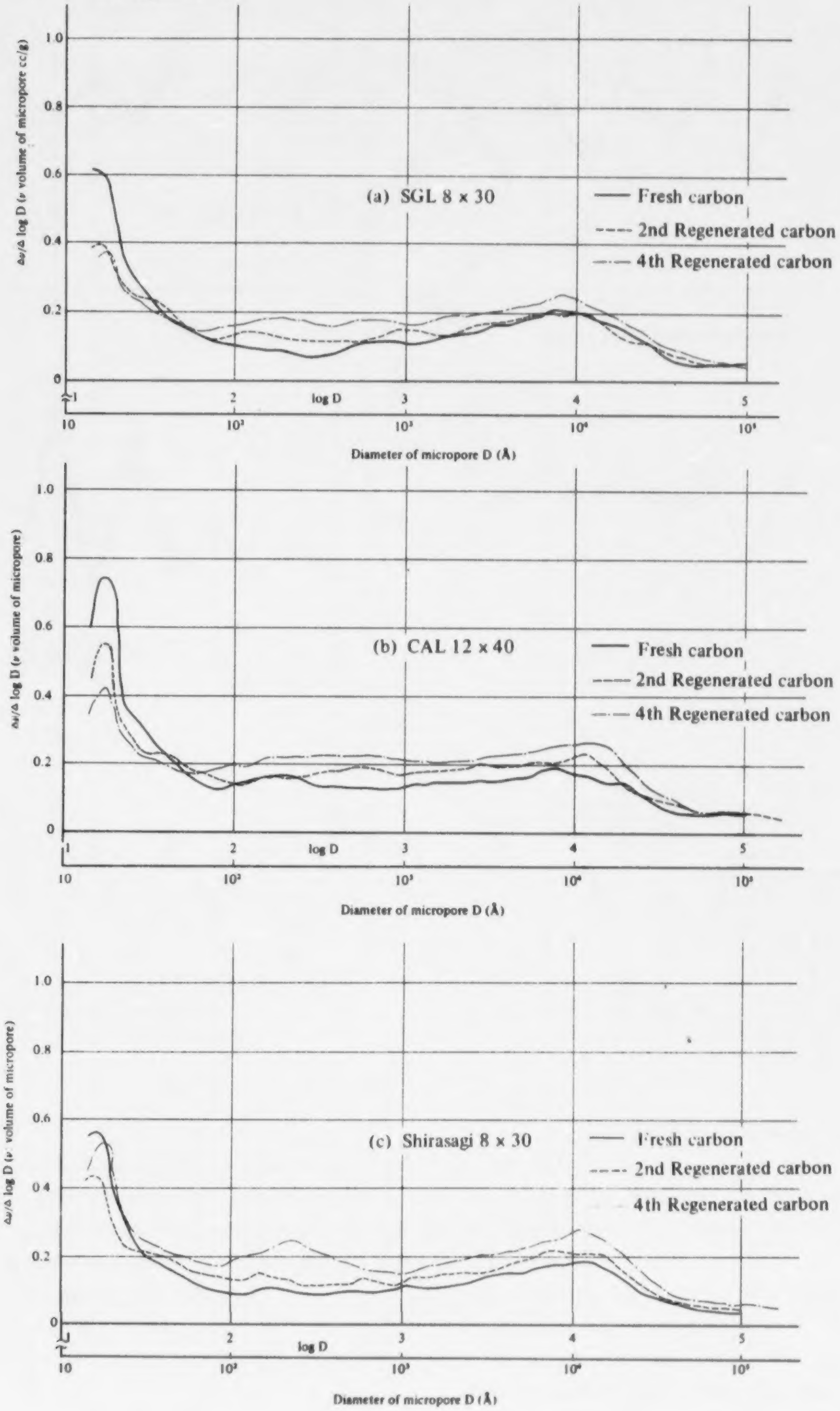
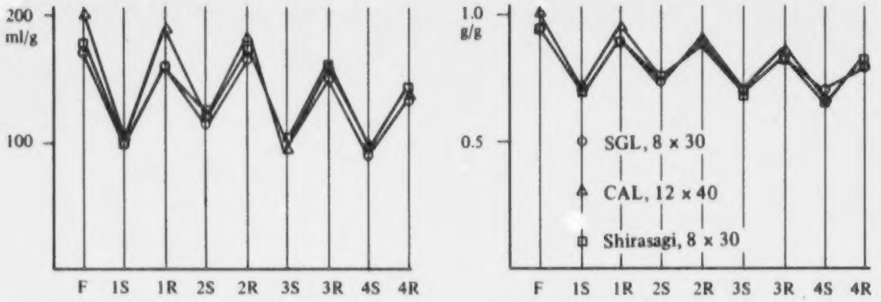
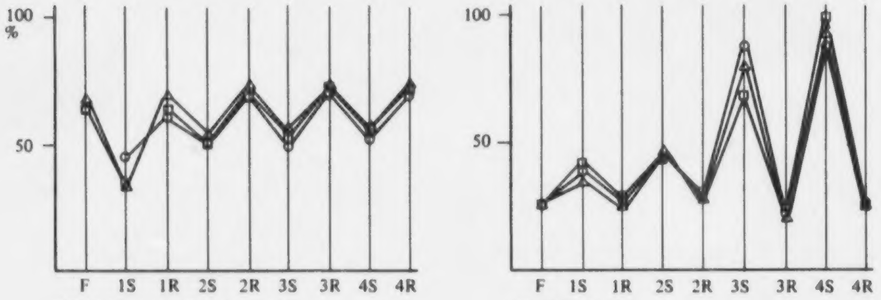


Fig. 4.6 Adsorption Characteristic Change of Granular Activated Carbons with Repeat of Regeneration



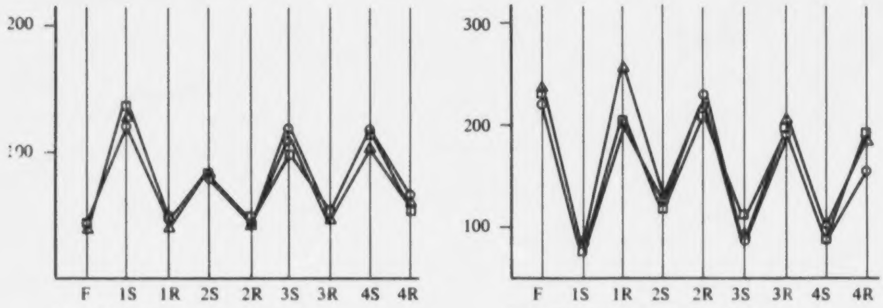
(a) Methylen blue decolorizing capacity

(b) Iodine adsorption capacity



(c) Molasses decolorizing capacity

(d) Phenole value



(e) ABS value

(f) Converse of ABS value (× 100)

ABS Value (Fig. 4.6 (e))

This value is an index for the amount of activated carbon required for the removal of the required ABS as in the case of phenole value. The fluctuating status is therefore much alike that shown in Fig. 4.6 (d). As it was difficult to compare it directly with physical characteristics, we took the converses in consideration of the meaning of the index and obtained the result as shown in Fig. 4.6 (f). From this diagram, it is found that the converses of ABS value fluctuates quite alike specific surface area, micropore volume of 12Å and more, methylene blue decolorizing capacity and iodine adsorption capacity.

4.2.4 ADSORPTION CHARACTERISTICS OF ORGANIC MATTER IN WATER

The major objective of wastewater treatment by activated carbon is to remove organic matter in the wastewater. In order to study the organic substances adsorption characteristics of activated carbon which is used for advanced wastewater treatment, it is considered important to measure the adsorption capacity of the carbon for same kind of organic substances as contained in wastewater. In this experiment, therefore, an experiment on isothermal adsorption has been carried out with TOC as an index for organic matter by using secondary effluent sampled from a pilot plant.

We have consolidated the data according to Freundrich equation which was obtained from the experiments, but due to insufficient data for each experiment and unstable property of secondary effluent taken for sample, we could not grasp a clear change of adsorption characteristics for organic substances with repeate of regeneration.

In order to analyze the adsorption characteristics of activated carbon against organic substances in wastewater, it is difficult to use the Freundrich equation because the concentration of the substances contained in secondary effluent is very low and varied. It is therefore felt necessary first to identify and measure the organic substances in secondary effluent to be treated and secondly to pick up some predominant organic substances and finally to study the adsorption characteristics with those pure substances.

4.2.5 DECLINE OF VARIOUS CHARACTERISTICS OF ACTIVATED CARBON

Various characteristics of activated carbon change little by little with repeat of use and regeneration, but will not be improved to the level of fresh carbon by regeneration. This is clearly understood from Fig. 4.3 and Fig. 4.6.

Now on the assumption that some value of characteristics will be increased A fold by every regeneration, the value of characteristics after regeneration n times, which is expressed in Pn, will be;

$$Pn = Po \cdot A^{n} \qquad \text{Log } Pn = n \log A + \log Po$$

where Po = Characteristics Value of Fresh Carbon

Here from the various characteristics values of fresh, spent and regenerated carbon obtained from the four times regeneration, we have gotten the values of A and Po by a minimum involution on the items which are likely to fit to the above equation and have gotten the result as shown in Table 4.4 (a). In the Table, r is a coefficient of correlation expressed in logarithms. Table 4.4 (b) indicates all C values taken from Table 4.4 (a). The degree of decline depend on the type of carbon. The degree of decline of various characteristics are lower than that of gravimetric recovery rate.

4.2.6 CORRELATION BETWEEN EACH CHARACTERISTIC OF ACTIVATED CARBON

As mentioned earlier in 4.2.3, there exists a greater correlation between physical characteristics and general adsorption capacity of activated carbons. For instance, specific surface area in Fig. 4.3 (d) and methylene blue decolorizing capacity in Fig. 4.6 (a) shows nearly the same fluctuation. We have then studied about correlation between various physical characteristics and general adsorption capacity

Table 4.4 (a) Coefficients which show the Characteristic Change with Repeat of Regeneration

$\log P_n = n \cdot \log A + \log P_o \quad A = 10^a, P_o = 10^b, C = (A' - 1) \times 100$

Item \ Reactor		1 ~ 2						3 ~ 4						5 ~ 6					
Item	Symbol	a	b	r	A	Po	C	a	b	r	A	Po	C	a	b	r	A	Po	C
Specific Surface Area m²/g		0.0295	2.980	0.9221	0.9343	955.0	△6.57	−0.0299	3.004	0.8789	0.9335	1009.3	△6.65	−0.0168	2.959	0.6769	0.9621	909.9	△3.79
Mean Pore Size of 0~300A Micropore		0.0195	1.321	0.9637	1.0459	20.94	4.59	0.0204	1.335	0.9680	1.0481	21.63	4.81	0.0207	1.317	0.9960	1.0488	20.75	4.88
Volume of Micropore	0~15μ cc/g	0.0102^{1}	0.0976	0.6129	1.0238	0.7987	2.38	0.0133^{2}	−0.0625	0.6578	1.0311	0.8660	3.11	0.0195^{4}	−0.1146	0.8742	1.0460	0.7681	4.60
	300A~15μ cc/g	0.0367^{1}	−0.5209	0.9347	1.0882	0.3014	8.82	0.0427^{7}	−0.4962	0.9784	1.1035	0.3191	10.35	0.0423	−0.5303	0.9799	1.1023	0.2949	10.23
	0~300A cc/g	-0.0103^{7}	−0.3008	0.6126	0.9764	0.5003	△2.36	-0.0105^{8}	−0.2615	0.5143	0.9759	0.5476	△2.41	0.0034^{8}	−0.3257	0.1987	1.0080	0.4724	0.80
	0~12A cc/g	-0.0410^{1}	−0.6971	0.9421	0.9098	0.2008	△9.02	-0.0426^{5}	−0.7035	0.9501	0.9065	0.1979	△9.35	-0.0437^{6}	−0.6777	0.9189	0.9041	0.2100	△9.59
Apparent Density g/ℓ		−0.0077	2.686	0.9318	0.9824	485.3	△1.76	−0.0067	2.655	0.9002	0.9847	451.6	△1.53	−0.0110	2.694	0.9787	0.9750	494.3	△2.50
Mean Particle Size mm		−0.02456	0.2154	0.9161	0.9450	1.642	△5.50	0.0082	0.0041^{4}	0.4499	1.0191	1.0096	1.91	-0.0033^{2}	0.2020	0.2225	0.9924	1.592	△0.76
Methylene Blue Decolorizing Capacity mℓ/g		−0.0228	2.237	0.9076	0.9489	172.6	△5.14	−0.0385	2.314	0.9762	0.9152	206.1	△8.48	−0.0188	2.248	0.8497	0.9576	177.0	△4.24
Iodine Adsorption Capacity g/g		−0.0175	-0.0263^{2}	0.9582	0.9605	0.9412	△3.95	-0.0232^{4}	-0.0005^{4}	0.9958	0.9479	0.9988	△5.21	-0.0158^{1}	−0.0275	0.9714	0.9643	0.9386	△3.57
Molasses Decolorizing Capacity %		0.0114	1.797	0.7418	1.0266	62.69	2.66	0.0090	1.825	0.7945	1.0210	66.90	2.10	0.0137	1.803	0.8745	1.0320	63.62	3.20
ABS Value		0.0340	1.635^{2}	0.8170	1.0814	43.17	8.14	0.0317	1.594	0.9061	1.0757	39.26	7.57	0.0193	1.6452	0.9396	1.0454	44.18	4.54
Converse of ABS Value		-0.0339^{7}	0.3642	0.8169	0.9248	2.313	△7.52	0.0317^{6}	0.4051	0.9072	0.9295	2.541	△7.05	−0.0162	0.3514	0.8728	0.9634	2.246	△3.66
Dry Weight of Carbon kg		-0.0710^{6}	2.9811	0.9915	0.8491	957.4	△15.09	-0.0682^{5}	2.9519	0.9791	0.8547	895.2	△14.53	-0.0499^{4}	2.9723	0.9922	0.8914	938.2	△10.86

n = time number of regeneration,
P_n = Value showing characteristic of the activated carbon after n th regeneration
P_o = Value showing characteristic of the fresh carbon
△ = sign of minus
r = Coefficient of correlation

Table 4.4 (b) Decline of Granular Activated Carbon Characteristics with Repeat of Regeneration

Items \ Name of Carbon		SGL, 8 x 30		CAL, 12 x 40		SHIRASAGI, 8 x 30	
Specific Surface Area		0.9343	6.57	0.9335	6.65	0.9621	3.79
Mean Pore Size of 0~300A Micropore		1.0459	−4.59	1.0481	−4.81	1.0488	−4.88
Volume of Micropore	0~15μ	1.0238	−2.38	1.0311	−3.11	1.0460	−4.60
	300A~15μ	1.0882	−8.82	1.1035	−10.35	1.1023	−10.23
	0~300A	0.9764	2.36	0.9759	2.41	1.0080	−0.80
	0~12A	0.9098	9.02	0.9065	9.35	0.9041	9.59
Apparent Density		0.9824	1.76	0.9847	1.53	0.9750	2.50
Mean Particle Size		0.9450	5.50	1.0191	−1.91	0.9924	0.76
Methylene Blue Decolorizing Capacity		0.9489	5.14	0.9152	8.48	0.9576	4.24
Iodine Adsorption Capacity		0.9605	3.95	0.9479	5.21	0.9643	3.57
Molasses Decolorizing Capacity		1.0266	−2.66	1.0210	−2.10	1.0320	−3.20
ABS Value		1.0814	−8.14	1.0757	−7.57	1.0454	−4.54
Converse of ABS Value		0.9248	7.52	0.9295	7.05	0.9634	3.66
Dry Weighs of Packed Carbon		0.8491	15.09	0.8547	14.53	0.8914	10.86

Note: Left number show recovery rate and right show declining rate (%). The numbers were calculated statistically from result of experiments.

as well as correlation among adsorption capacities.

Assuming that item Y and item X is in the relationship as Y = AX + B, we have obtained the values of A and B as well as coefficient of correlation R as shown in Table 4.5 (a). Further in order to make it easier to understand, we have expressed these values in matrix as shown in Table 4.5 (b). In Table 4.5 (a) and Table 4.5 (b), coefficient of correlation marked with double circles are 0.95 and above, coefficient of correlation marked with a single circle 0.85 to 0.95, coefficient of correlation marked with a triangle 0.70 to 0.85.

From the Table 4.5 (b), relatively high correlation can be noticed as follows;

(1) Micropore volume from 0 to 300Å vs. specific surface area and total micropore volume.

(2) Micropore volume from 30 to 60Å vs. total micropore volume from 0 to 15μ.

(3) Methylene blue decolorizing capacity vs. specific surface area, micropore volume from 0 to 300Å, micropore volume from 12 to 30Å and micropore volume from 0 to 12Å.

(4) Iodine adsorption capacity vs. specific surface area, micropore volume from 0 to 12Å and methyle blue decolorizing capacity.

(5) Molasses decolorizing capacity vs. Micropore from 0 to 300Å

(6) The converses of ABS value vs. specific surface area, micropore volume from 0 to 300Å, micropore volume from 0 to 12Å, methylene blue decolorizing capacity and iodine adsorption capacity.

4.2.7 PARAMETER AS A BASE FOR CALCULATION OF REGENERATING COST

The regeneration cost of activated carbon is a very expensive so, as the case may be, it is advisable to purchase fresh carbon, but in case of regeneration, it must be more in-expensive than purchasing fresh carbon if we can enhance recovery rate by reducing the costs for fuel, for electricity and for labor since we require no raw materials. Out of the three costs above, most important one is considered to be fuel cost so we have prepared Table 4.6 covering unit values of all the relevant items of fuel costs from the data obtained from this experiment. The moisture of spent carbon feeded into a regeneration furnace in this experiment was 40 to 50%, but from Table 4.6, propane gas consumption for spent carbon with such water content was 50 to 55 ℓ/kg wet weight of feeded carbon, 87 to 99 ℓ/kg dry weight of feeded carbon while fuel oil consumption at after-burning room for treatment of exhaust gas was 0.1 to 0.14 ℓ/kg dry weight of feeded carbon. Further, the feeding rate of

Table 4.5 (a) Correlation between Each Characteristic of Granular Activated Carbon

Y = AX + B

		Carbon	SGL			CAL			Shirasagi		
	X	Y	A	B	R	A	B	R	A	B	R
△	Specific Surface Area m^2/g	Volume of Total Micropore cc/g	0.0005746	0.3148	0.8285	0.0006005	0.3652	0.8487	0.0005522	0.3706	0.7608
◎	Specific Surface Area m^2/g	Volume of 0~300A Micropore cc/g	0.0004762	0.07339	0.9676	0.0004919	0.08239	0.9738	0.0004899	0.06442	0.9535
○	Specific Surface Area m^2/g	Methylene Blue Decolorizing Capacity mℓ/g	0.1506	27.60	0.9186	0.1646	27.13	0.9200	0.1634	22.99	0.9272
○	Specific Surface Area m^2/g	Iodine Adsorption Capacity g/g	0.0004774	0.4668	0.9484	0.0004848	0.4711	0.9481	0.0004969	0.4499	0.9009
○	Specific Surface Area m^2/g	Converse of ABS Value (%)	0.002861	−0.4466	0.9036	0.002842	−0.3500	0.9328	0.003073	−0.5582	0.9448
○	Volume of Total Micropore cc/g	Volume of 0~300A Micropore cc/g	0.6644	−0.08849	0.9364	0.6744	−0.1025	0.9447	0.6470	−0.08314	0.9141
◎	Volume of Total Micropore cc/g	Volume of 30~60A Micropore cc/g	0.06289	0.006752	0.9596	0.06782	0.005039	0.9317	0.07311	0.002073	0.9771
△	Volume of 300A~15μ Micropore cc/g	Molassis Decolorizing Capacity %	121.6	17.85	0.7013	150.6	7.446	0.7549	168.6	2.293	0.7609
○	Volume of 0~300A Micropore cc/g	Methylene Blue Decolorizing Capacity mℓ/g	293.2	13.67	0.8798	301.5	13.77	0.8510	289.5	19.37	0.8439
○	Volume of 0~300A Micropore cc/g	Molassis Decolorizing Capacity %	88.57	23.05	0.8794	95.00	20.76	0.8485	110.1	14.80	0.8834
○	Volume of 0~300A Micropore cc/g	Converse of ABS Value (%)	5.633	−0.7365	0.8753	5.496	−0.7054	0.9114	5.679	−0.7214	0.8971
○	Volume of 12~30A Micropore cc/g	Methylene Blue Decolorizing Capacity mℓ/g	832.8	28.03	0.8576	835.5	26.08	0.8980	921.2	21.93	0.8446
△	Volume of 12~30A Micropore cc/g	Iodine Adsorption Capacity g/g	2.677	0.4635	0.8983	2.488	0.4642	0.9358	2.649	0.4656	0.7762
△	Volume of 12~30A Micropore cc/g	Molassis Decolorizing Capacity %	216.2	31.79	0.7368	219.1	30.83	0.7453	333.8	17.84	0.8422
○	Volume of 0~12A Micropore cc/g	Methylene Blue Decolorizing Capacity mℓ/g	680.6	38.65	0.9186	788.0	42.56	0.9250	607.5	54.05	0.9224
○	Volume of 0~12A Micropore cc/g	Iodine Adsorption Capacity g/g	2.175	0.4995	0.9565	2.336	0.5146	0.9596	1.932	0.5328	0.9373
○	Volume of 0~12A Micropore cc/g	Converse of ABS Value (%)	12.75	−0.2121	0.8913	13.36	−0.05296	0.9218	11.34	−0.03731	0.9331
◎	Methylene Blue Decolorizing Capacity mℓ/g	Iodine Adsorption Capacity g/g	0.002993	0.4026	0.9754	0.002829	0.4078	0.9903	0.003059	0.3775	0.9775
◎	Methylene Blue Decolorizing Capacity mℓ/g	Converse of ABS Value (%)	0.01916	−0.9927	0.9923	0.01644	−0.6996	0.9657	0.01817	−0.9035	0.9845
◎	Iodine Adsorption Capacity g/g	Converse of ABS Value (%)	6.103	−3.331	0.9699	5.644	−2.935	0.9475	5.674	−2.934	0.9621

Table 4.5 (b) Correlation between Each Characteristic of Granular Activated Carbon

			Converse of ABS Value	Molasses Decolorizing Capacity	Iodine Adsorption Capacity	Methylene Blue Decolorizing Capacity	Volume of Micropore					
							0~12A	12~30A	30~60A	0~300A	300A~15μ	Total Micropore 0~15μ
Specific Surface Area		m²/g	90~95 ○	–	90~95 ○	91~93 ○	–	–	–	95~98 ◎	–	76~85 △
Volume of Micropore	Total Micropore 0~15μ	cc/g	–	–	–	–	–	–	93~98 ◎	91~95 ○	–	
	300A~15μ	cc/g	–	70~77 △	–	–	–	–	–	–		
	0~300A	cc/g	87~92 ○	84~89 ○	–	84~88 ○	–	–	–			
	30~60A	cc/g	–	–	–	–	–	–				
	12~30A	cc/g	–	73~85 △	77~94 △	84~90 ○	–					
	0~12A	cc/g	89~94 ○	–	93~96 ○	91~93 ○						
Methylene Blue Decolorizing Capacity			96~100 ◎	–	97~100 ◎							
Iodine Adsorption Capacity			94~97 ◎	–								
Molasses Decolorizing Capacity			–									

Numbers in the frame show range of correlation coefficient (x 100).

steam for reactivation of carbon was 0.7 to 1.0 kg/kg while air blow rate for gas burning was approximate 21.1 liter per liter of propane gas.

Mean temperature distribution of atmosphere in furnace during regeneration is as shown in Fig. 4.7 and the temperature was controlled satisfactorily.

The concentration of CO, CO_2 and O_2 for atmospheric gas in the lowest stage which we analyzed is as shown in Table 4.7 and from this data, it is found that the reductive atmosphere was obtained.

4.2.8 OTHERS

In this experiment, component of exhaust gas, concentration of odor substances, and the fluctuation of content of inorganic substances in activated carbon before and after regeneration was measured.

Properties and Component of Exhaust Gas

Properties and component of exhaust gas from regenerating furnace itself, after-burning room and scrubber in regeneration of activated carbon are as shown in Table 4.8. From Table 4.8, it can be said that the dusts and odor strength are removed by after-burning, but sulphurous acid gas and NOx rather increase. Scrubber is effective to remove dusts and sulphurous acid gas, but did not contributed to the removal of NOx. The increase of odor strength at scrubber was due to the use of secondary effluent for scrubbing.

Concentration of Odor Substances

In our second and third regeneration, the concentrations of odor substances and dangerous substances contained in exhaust gas from regenerating furnace and after-burning room were measured. The result is shown in Table 4.9. From Table 4.9, it can be said that the concentration of odor substances at the outlet of regeneration furnace is considerably high, but after after-burning, such concentration was considerably reduced. This explains the favorable impact of after-burning to odor substances.

Fluctuation of Inorganic Substances Content with Repeat of Regeneration

Inorganic substances contained in carbon and fresh carbon before and after regeneration were measured and the fluctuation of content with repeat of regeneration was studied. Items measured were altogether nine covering calcium, cadmium, chromium, copper, mercury, magnesium, manganese, lead and zinc. The results of this measurement are as shown in Table 4.10. With the exception of Ca, Cd, Mn,

Table 4.6 Basic Data on Fuel Cost

Item	Reactor Number Regeneration	1	2	3	4	5	6
Consumption Rate of Propane Gas to Wet Weight of Feeded Spent Carbon (ℓ/kg WSC)	1st	48.3	47.7	49.0	43.4	44.3	43.4
	2nd	57.8	57.6	54.8	51.1	55.2	55.6
	3rd	49.7	52.3	62.3	54.4	63.4	51.8
	4th	55.5	53.4	53.1	49.6	62.1	50.7
	Mean	52.8	52.8	54.8	49.6	56.3	50.4
Consumption Rate of Propane Gas to Dry Weight of Feeded Spent Carbon (ℓ/kg DSC)	1st	80.0	82.6	85.2	78.9	77.1	75.9
	2nd	95.2	101.3	94.4	93.4	93.3	93.9
	3rd	89.0	99.0	117.0	103.7	109.1	92.7
	4th	96.3	97.1	97.9	100.9	106.3	93.5
	Mean	90.1	95.0	98.6	94.2	96.5	89.0
Oil Consumption Rate for After Burning to Wet Weight of Feeded Spent Carbon (ℓ/kg WSC)	1st	0.0832	0.107	0.107	0.121	0.0967	0.116
	2nd	0.144	0.163	0.145	0.151	0.121	0.161
	3rd	–	0.123	0.100	0.148	0.0946	0.120
	4th	0.121	0.133	0.112	0.141	0.0971	0.120
	Mean	0.116	0.132	0.116	0.140	0.102	0.129
Oil Consumption Rate for After Burning to Dry Weight of Feeded Spent Carbon (ℓ/kg DSC)	1st	0.138	0.185	0.185	0.220	0.168	0.202
	2nd	0.237	0.287	0.249	0.277	0.204	0.216
	3rd	–	0.233	0.189	0.281	0.163	0.230
	4th	0.209	0.242	0.206	0.272	0.166	0.221
	Mean	0.195	0.237	0.207	0.263	0.175	0.217
Steam Feeding Rate to Dry Weight of Feeded Spent Carbon (kg/kg DSC)	1st	0.737	0.962	0.919	0.941	0.870	0.864
	2nd	0.535	1.006	0.899	0.919	0.819	0.823
	3rd	0.901	1.013	1.199	1.052	0.826	0.951
	4th	0.875	0.909	0.926	0.993	0.801	0.867
	Mean	0.762	0.973	0.986	0.976	0.829	0.876
Air Blowing Rate to Propane Gas (N-ℓ/N-ℓ G)	1st	22.5	24.6	23.9	25.3	25.1	25.0
	2nd	20.2	18.9	18.4	19.0	18.1	17.2
	3rd	22.7	21.5	20.9	20.8	20.4	21.7
	4th	21.6	19.6	21.6	20.5	21.0	22.3
	Mean	21.8	21.2	21.1	21.4	21.2	21.6

Fig. 4.7 Distribution of Temperature in Furnace

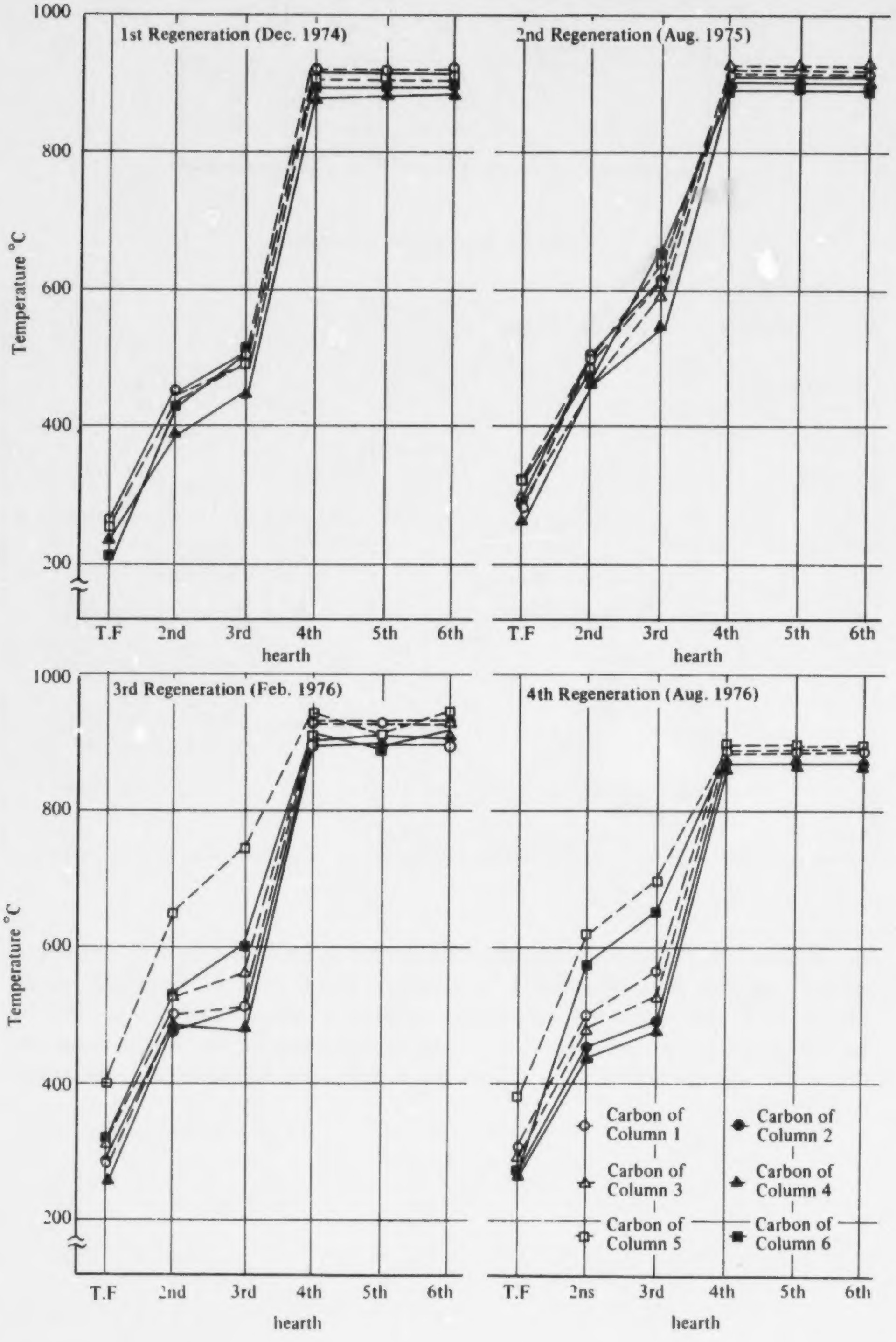


Table 4.7 Contents of Atmosphere Gas (%)

Item Times of Regeneration	CO	CO_2	O_2
1	2.0~3.4	12.0~12.2	0.1 or less
2	1.9~3.4	11.9~12.9	0
3	1.7~1.9	11.0~11.8	0
4	1.7~1.9	11.1~11.6	0

Table 4.8 Exhaust Gas Component

Items	Method of Analysis	Outlet of Regeneration Furnace			Outlet of After Burning Room			Outlet of Scrubber		
		1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd
Gas Temperature °C		240	240	266	620	650	643	40	50	25
Dry Gas Volume Nm^3/H	JIS Z8808	69	83	88	675	272	185	373	294	229
Moisture V/V %	JIS Z8808	42.4	42.8	39.4	22.5	21.6	35.9	2.4	12.2*	13.1
Dust g/Nm^3	JIS Z8808 Dust Tube Method	1.31	3.14	3.84	0.16	0.27	0.26	0.05	0.05	0.06
SO_2 ppm	JIS K0103 Solution Conductivity Method	3	<5	<5	235~240	140~170	350~400	6~9	<5	<5
NO ppm	JIS K0104 Chemical Radiation Method	62~74	66~70	–	100~110	85~89	–	113~123	107~111	–
NOx ppm	JIS K0104	65~76	66~70	40~60	105~115	88~94	125~135	118~128	107~115	125~130
Power of Odor (PO)	Equilibrium Method in Salt Water	11.9	11.9	10.4	2.6	1.0	0	5.3	2.3	5.1
CO_2 V/V %	Orsat Method	10.9	10.4	9.6	11.4	7.6	10.2	8.3	5.8	9.9
O_2 V/V %	Orsat Method	3.3	3.3	0.9	4.0	9.3	6.1	8.6	12.4	6.2
CO V/V %	Orsat Method	–	3.3	2.9	–	0	0	–	0	0.2

and Pb, the content of each substance was greater in spent carbon than regenerated carbon and also normally greater in primary reactor than in secondary reactor. This explains that activated carbon has adsorbed metals to some extent and has accumulated part of metals without taking off it in regeneration. A high concentration of Ca was due to the impact from the regeneration of lime in the regenerating reactor.

Table 4.9 Concentration of Odor and Dangerous Substances in Exhaust Gas (ppm)

Items		Measuring Method	Threshold Value	Outlet of Furnace		Outlet of After Burning Room	
Name	Chemical Formula			2nd	3rd	2nd	3rd
Hyarogen Cyanide	HCN	JIS K0109 Pyridine Pyrazolon Method		4.2	5.2	<0.08	<0.006
Carbon Disulfide	CS_2	E.A. Notification No.9 FPD·GC Method		120	90	<0.1	<0.02
Acetic Acid	CH_3COOH	FID·GC Method		280	370	40	<0.2
Formaldehyde	$HCHO$	JIS K0102 Acetyleace-tone Method	1.0	<0.7	0.19	<0.5	<0.01
Acetaldehyde	$CH_3 \cdot CHO$	FID·GC Method	0.21	<30	19	<30	<0.3
Methane	CH_4	FID·GC Method		1100	1.360	<10	35
Ethane	C_2H_6	FID·GC Method		54	3.5	<10	<0.3
Ammonia	NH_3	JIS K0099 Indo Phenol Test		990	680	0.49	0.25
Tri-methyl Amine	$(CH_3)_3 \cdot N$	E.A. Notification No.9	0.0021	<0.05	<0.08	<0.05	<0.008
Hydrogen Sulfide	H_2S	E.A. Notification No.9	0.005	<3	780	<0.03	0.15
Methyl Mercaptan	$CH_3 \cdot SH$	E.A. Notification No.9	0.041	<1	<2	0.002	<0.05
Dimethyl Sulfide	$(CH_3)_2 \cdot S$	E.A. Notification No.9		<0.5	<0.1	<0.001	<0.03

Table 4.10 Change of Inorganic Matter Contents with Repeat of Regeneration

Unit: ppm

Car-bon	Reactor / Status of Carbon	Primary									Secondary								
		Ca	Cd	Cr	Cu	Hg	Mg	Mn	Pb	Zn	Ca	Cd	Cr	Cu	Hg	Mg	Mn	Pb	Zn
SGL	F	40	<0.5	<0.5	0.5	(0.1)	69	3	<5	7	40	<0.5	<0.5	0.5	(0.1)	69	3	<5	7
	1S	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	1R	825	<0.5	12	16.5	0.02	131	82	<5	20.5	–	–	–	–	–	–	–	–	–
	2S	650	<0.5	25	41.0	1.47	134	54.5	<5	120	335	<0.5	28	43.0	0.28	130	104	<5	124
	2R	645	<0.5	10	31.0	0.05	118	62	<5	8.5	278	<0.5	8	19.5	0.04	100	77	<5	9
	3S	1575	<0.5	12.1	45.7	0.10	194	52.3	<5	132	855	<0.5	9.7	37.1	0.13	149	68	<5	144
	3R	1260	<0.5	6.7	25.7	<0.01	148	87.4	<5	7.5	995	<0.5	7.3	20.0	<0.01	144	106.1	<5	10.0
	4S	1800	<0.5	25	49	0.02	260	82	<5	223	915	<0.5	15	31	0.04	161	103	<5	206
	4R	2100	<0.5	15	39	<0.01	244	105	<5	18	1270	<0.5	11	16	<0.01	171	127	<5	17
CAL	F	59	<0.5	<0.5	1.5	(0.11)	48	5	5	2	59	<0.5	<0.5	1.5	(0.11)	48	5	5	2
	1S	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	1R	775	<0.5	24	37.5	0.03	141	195	<5	21.0	–	–	–	–	–	–	–	–	–
	2S	725	<0.5	40	55.0	1.50	139	73	10	137	370	<0.5	37	39.5	0.15	135	162	<5	139
	2R	755	<0.5	24	34.0	0.03	129	99	<5	13.5	330	<0.5	11	14.5	0.04	111	125	<5	11
	3S	2115	<0.5	14.6	61.7	0.06	270	88.4	<5	156	1260	<0.5	12.8	42.8	0.10	198	99.0	<5	159
	3R	1880	<0.5	12.1	49.1	0.01	244	156.5	<5	9.0	950	<0.5	9.1	14.3	<0.01	249	140.0	<5	7.0
	4S	2560	<0.5	28	70	0.10	385	140	<5	284	1235	<0.5	22	34	0.05	204	172	<5	206
	4R	2240	<0.5	17	44	<0.01	328	156	<5	24	1200	<0.5	13	14	<0.01	174	150	<5	16
SHIRASAGI	F	36	<0.5	<0.5	0.5	(0.09)	67	2.5	<5	4.5	36	<0.5	<0.5	0.5	(0.09)	67	2.5	<5	4.5
	1S	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	1R	850	<0.5	12	23	0.02	135	86	<5	10.5	–	–	–	–	–	–	–	–	–
	2S	655	<0.5	29	49	0.92	131	76	<5	126	388	<0.5	25	31.5	0.27	126	105	5	122
	2R	605	<0.5	10	22	0.02	119	72	<5	9	273	<0.5	7	10.5	0.01	102	76	<5	8.5
	3S	2125	<0.5	11.5	88.5	0.20	219	124.5	<5	122	790	<0.5	11.5	20.7	0.25	132	88.3	<5	145
	3R	1125	<0.5	4.8	17.7	<0.01	135	105.4	<5	7.0	750	<0.5	6.1	8.6	<0.01	120	108.5	<5	12.0
	4S	2505	<0.5	11	64	0.10	309	157	<5	247	1215	<0.5	13	30	0.07	188	212	<5	156
	4R	2225	<0.5	11	37	<0.01	269	192	<5	26	1200	<0.5	8	7	<0.01	157	149	<5	8.5

The numbers in ( ) are not exact.

4.3 SUMMARY

Repeat of regeneration experiments have been conducted on three types of activated carbon being used for an advanced wastewater treatment at Kyoto pilot plant and the following have been confirmed;

(1) Recovery rate was 90 to 100% gravimetrically but 85 to 95% volumetrically.

(2) Through the repeated regeneration, apparent density and micropore volume was decreased, but mean particle size of micropore from 0 to 300Å and micropore volume from 300Å to 15 μ was increased. The rest of the items have not been changed so much.

(3) Out of general adsorption capacity, methylene blue decolorizing capacity and iodine adsorption capacity were decreased through the repeat of regeneration while molasses decolorizing capacity was slightly increased. Phenole value and ABS value were not fluctuated so much.

(4) It was found that the following items had extremely high correlation each other;

Micropore volume from 0 to 300Å vs. specific surface area, micropore volume from 30 to 60Å vs. total micropore volume from 0 to 15 μ. Iodine adsorption capacity vs. methylene blue decolorizing capacity, the converses of ABS values vs. methylene blue decolorizing capacity and iodine adsorption capacity.

(5) Propane gas consumption by dry weight of feeding activated carbon was 87 to 99 ℓ/kg. Fuel oil consumption for after-burning was 0.1 to 0.14 ℓ/kg. Steam injection quantity for reactivation of carbon was 0.7 to 1.0 kg/kg.

(6) Due to after-burning, dust contained and odor strength of exhaust gas was decreased, but SOx and NOx were increased. Scrubber was effective for the reduction of SOx, but not for NOx.

(7) Activated carbon adsorbes or removes metals in wastewater to some degree.

ACKNOWLEDGEMENT

The author thanks Mr. Yoneda and his stuff of sewage Works Bureau of Kyoto City for their cooperation in the experimental study.

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DEVELOPING PROGRAMS FOR CONTROL OF NONPOINT (DIFFUSE) SOURCES OF WATER POLLUTION

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ABSTRACT

Controlling nonpoint sources of pollution will be necessary in order to meet the goals of the Federal Water Pollution Act. The paper describes the magnitude of the pollution from these sources and describes a new approach being tried to solve the problem. It primarily relies on State and local governments prescribing "best management practices" and making important land use decisions. The program is highlighted by active citizen participation and a shift from voluntary compliance to at least a semi-regulatory format.

INTRODUCTION

Non-point sources of water pollution, such as runoff from croplands, urban stormwater, and strip mining, are becoming the single most important water quality problems. To help solve this pollution problem the U.S. Congress placed primary responsibility on the States and local units of government. The wastewater management planning process being carried out under Section 208 of the 1972 Federal Water Pollution Control Act offers some good possibilities to control this aspect of water pollution.

BACKGROUND

In the U.S.A. current estimates indicate that perhaps half our national water pollution problem may come from nonpoint (diffuse) sources of pollution. These sources generally involve runoff, usually after storms, from man-distrubed land, streets, parking lots and other surface areas. The seriousness of the problem is highlighted by the fact that it is predicted the 1983 goals of the Federal Water Pollution Control Act--i.e. all waters being clean enough for fishing and swimming --will not be met unless this source of pollution is brought under control.

Further highlights of the problems associated with nonpoint source pollution are: (1) Two billion tons of sediment are delivered to lakes and streams annually from over 400 million acres of croplands, as well as large amounts of nitrogen from fertilizers, phosphorus from nonpoint sources, animal wastes from feedlots, and toxic pesticides.

(2) Between 5 and 10 percent of the total sediment load is estimated to come from the 10 to 12 million acres of Commercial forest harvested each year.

(3) Strip mining, which affects about 350,000 acres annually, results in the discharge of millions of tons of acidity and sediment.

(4) Urban sprawl, which consumes hundreds of square miles per year, generates sediment at an even greater rate than agricultural activities.

(5) The runoff of stormwater in urban areas accounts for the pollution of waters with large amounts of toxic and oxygen--demanding materials. It also contains large amounts of suspended sediments and extremely high but variable coliform counts.

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As point sources of pollution--sewage and industrial wastes discharged through pipes--are reduced, the nonpoint sources gain in relative importance. There hasn't been a systematic national program for monitoring nonpoint source pollution and its water quality impact-but-most everyone agrees this pollution source is a major problem.

NEW APPROACH

One new and important tool for controlling nonpoint source pollution is Section 208 of the Federal Act. It calls for State and regional planning for water quality management. Funds to do this planning are provided by EPA. As in other areas, the Federal Government is providing an incentive and mechanism for local governments to deal with a growing problem.

This planning for controlling nonpoint sources of pollution encourages States and local government agencies to prescribe so-called best management practices (BPT). We define BPT as a practice, or combination of practices, that is determined by a State or local governmental agency to be the most effective and practicable means of preventing or reducing the amount of pollution generated by nonpoint sources to a level compatible with water quality goals. The term practicable includes technological, economic and political considerations. BPT's are determined after assessment of the problem, examination of alternative practices and appropriate public participation. These practices may vary widely according to local climate, topography, soils, geology, vegetation and other conditions.

For the most part, best management practices are known and many are in daily use. But these techniques are not being widely applied in many areas where serious water quality degradation is occurring.

Studies have demonstrated that good conservation practices can reduce sediment yield anywhere from 50 to 90 percent. It has been estimated that if soil conservation practices were applied to all farm land, nearly 50 percent of the sediment loading in streams would be eliminated, in addition to a reduction in such related pollutants as nutrients and pesticides.

Some typical best management practices we hope will be adopted through the 208 planning process are as follows:

(1) Contour farming to reduce erosion potential; (2) leaving buffer strips of trees and shrubs along streams to reduce soil erosion and to reduce temperature increases from sunshine; (3) frequent street sweeping during dry periods to prevent storm water from carrying debris, oil and other material into storm sewers; (4) sediment traps at new construction sites and at shopping center locations to prevent siltation of streams; (5) grass seeding or other vegetation planting to stabilize areas of construction activity; and (6) use of culverts and other engineering structures to control water flow and thereby reduce erosion. There are many other examples of practices which can be effective. New techniques are continually being developed which should be put to work. Most important is that each water quality management agency define its own set of such practices, tailored to meet specific local problems and environmental conditions.

Prevention is the key to the control of nonpoint source pollution. And, in this regard, intelligent use of the land is most important. Water quality is affected, often very significantly, by land use decisions. For part of the prevention answer therefore we are expecting local governments to use land zoning to reduce runoff by precluding development in ground water recharge areas, flood plains and ecologically sensitive areas. Public acquisition of land for preservation areas is also encouraged as is preferential tax treatment to retain land for open space or agricultual purposes.

IMPLEMENTATION

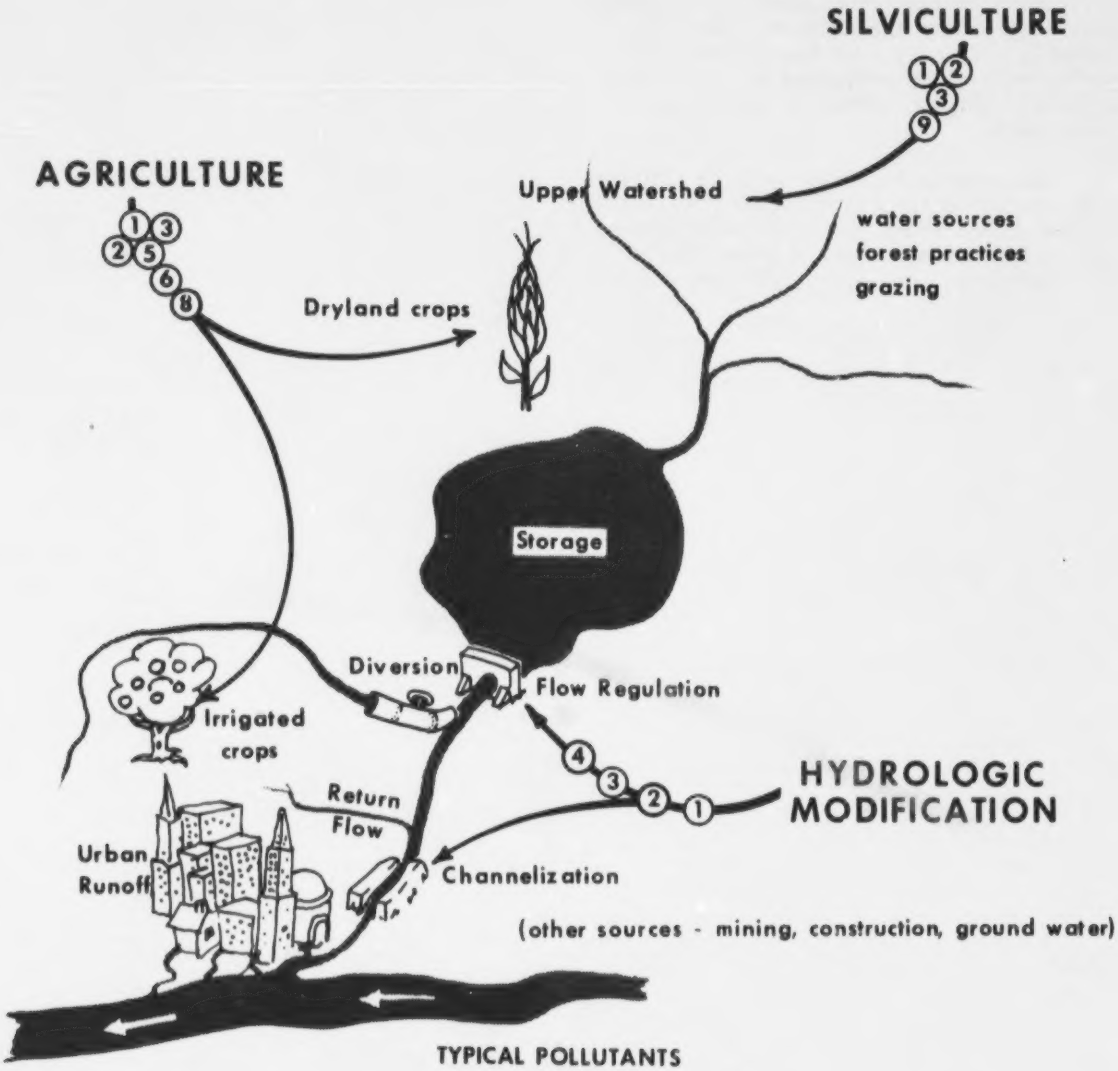
There already exist many Federal, State and local programs that deal with some forms of nonpoint source pollution. The Federal Water Pollution Control Act, and particularly Section 208, seeks to significantly expand on and coordinate these programs. Also, the Act suggests the programs be developed and carried out with the help of a major public participation effort. This has resulted in each State and local government doing the nonpoint source control planning to appoint advisory committees to provide assistance.

The final result will be a workable plan that identifies a governmental unit best able to perform the necessary tasks.

It will also often contain regulations and ordinances controlling septic tank pollution; land used for development; and prescribing best management practices for agriculture, logging, and overall construction activity. By putting these measures into the form of regulations and ordinances, a semi-regulatory (enforcement) cast is given to the program. This is probably necessary even though nonpoint pollution will be prevented or eliminated primarily through education and voluntary compliance.

The program described will be continually evaluated for its effectiveness. If an evaluation shows the goals aren't being met additional measures, such as tougher best management practices, will be adopted.

THE NONPOINT SOURCES PROBLEM



TYPICAL POLLUTANTS

1 Sediment
2 Nutrients
3 Temperature
4 Dissolved Gases - O2, N2
5 Pathogens
6 Pesticides
7 Heavy Metals
8 Dissolved Salts
9 Organic Materials

FIGURE 1

HISTORY OF THE CONSTRUCTION GRANTS PROGRAM

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ABSTRACT

The 1976 Needs Survey projected that by the year 1990, 34% of the United States population will be served by secondary treatment and 55% by greater than secondary treatment. Additional funding over the available $18 billion will be required -- $13 billion for secondary, $21 billion for greater than secondary, and $18 billion for new interceptor sewers. This paper discusses the types of treatment systems that are being, or will be, employed in the EPA Construction Grants Program and the factors that affect their selection. Particular attention is given to new technologies. The grants process, employing the concept of cost-effectiveness, is discussed. In addition to the 1976 Needs Survey, information for this paper was gathered by interviews with representatives from the ten EPA Regional offices and the review of over 1000 projects which employ new technologies. The survey showed the two key factors which impacted the implementation of a new technology in the Construction Grants Program were its cost-effectiveness and the level of confidence associated with the use of the system.

INTRODUCTION

The United States Environmental Protection Agency is primarily a regulatory organization which was established in 1970. However, included in the responsibilities of the USEPA is the management of a public works program for the construction of publicly owned wastewater treatment plants. With $18 billion in total Federal funding, the Construction Grants Program is one of the Nation's two largest public works programs, second only to the federal highways program in terms of size.

The EPA Construction Grants Program was authorized under the Federal Water Pollution Control Act, as amended in 1972 (Public Law 92-500). This act is the fourth in a series, the first of which was passed in 1948. The major public works program began its effective life in 1956 with annual appropriations limited to $50 million. Originally, individual grants were limited to 30% of the cost and were not to exceed $250,000. Although the funds were limited, this program did represent the beginning of Federal recognition that funding aid would be necessary for the construction of municipal wastewater treatment systems.

Recognizing the increasing threat of water pollution caused by insufficient wastewater treatment and the lack of communities to fund such projects, the Congress in 1966 passed a new law (PL 84-660). The Federal level of participation was increased to a range between 40-55%. Appropriations increased from about $200 million in 1968 to $1 billion in 1971. Under the grants program authorized by PL 84-660, nearly 14,000 grants for more than 5.2 billion Federal dollars were made.

In the late 60's, water quality standards, exposure of the public to water pollution issues and resultant enforcement conferences prompted municipalities and industries to consider action toward reducing the pollution load in their discharges. The greatest input to the improvement of the nation's water came with the passage of the Federal Water Pollution Control Act (PL 92-500) in October 1972. The primary goal of PL 92-500 is to "restore and maintain the chemical, physical and biological integrity of the Nation's waters."

National effluent limitations were required for all municipal wastewater treatment plants and were to be implemented in two increments. Initially, municipal wastewater treatment facilities must attain a minimum of secondary treatment by July 1, 1977. EPA has defined secondary treatment, based on a monthly average, as 30 mg/l of biochemical oxygen demand and suspended solids and a pH between 6.0 and 9.0. In the second level of treatment, all publicly owned treatment works must achieve "best practicable waste treatment technology" (BPWTT) by July 1, 1983. Because of budgetary constraints, EPA has established secondary treatment as the minimum criteria for BPWTT for those plants that are not required to meet more stringent water quality standards. The recently completed EPA Needs Survey showed that 47% of the facilities will require a level of treatment greater than secondary effluent. In these cases, additional BOD, suspended solids, nitrogen and phosphorus removals may be required.

In order to attain these goals, authorization for the Construction Grants Program was included within PL 92-500. The Federal share for the cost of the treatment systems is 75% of the eligible costs. Localities and many States share the remaining capital costs. Industrial users are required to pay the capital costs for the portion of the treatment works capacity they use. Operation and maintenance costs are shared by the local users. Projects are selected for funding based on the States' priority lists. States are required to revise their lists annually based on national and State criteria, severity of the pollution problem, existing population and the need to preserve high quality waters.

The grants process is divided into the following three segments:

Step 1 - Facility planning. Cost-effective and environmentally sound projects are developed through consideration of alternatives for treatment plant size, site, type of process, method of effluent and sludge disposal, interceptor sewer routing, etc. Infiltration/Inflow analyses and sewer systems survey requirements must be met; environmental assessments, public input, and consideration of all the other applicable Federal laws are required.

Step 2 - Design & Specifications Preparation. Preliminary development of a proportionate user charge system and industrial cost recovery must begin. The end product is a set of detailed drawings, specifications and cost estimates suitable for bidding and construction.

Step 3 - Construction. Bids must be solicited and reviewed and contracts awarded; on-site inspection, progress payments and audits are made according to EPA regulations.

As of January 1977, $12.2 billion of the authorized $18 billion has been obligated to projects. Projects which have received funding include: approximately 4500 Step 1 facilities plans for $350 million, 1200 Step 2 designs for $350 million, and 2300 Step 3 construction projects for $10.5 billion. As seen, with 5700 projects yet to be funded for construction and more projects remaining on State priority lists, additional funding will be necessary.

The results of the 1976 Needs Survey were submitted to Congress on February 9, 1977. In order to serve the sanitary sewer and treatment needs of the 1990 population, EPA has estimated that approximately $52 billion will be required. EPA has recommended to Congress that funds be authorized over a 10 year period. The cost of $52 billion for the construction of municipal wastewater treatment plants can be divided into $13 billion for secondary treatment, $21 billion for greater than secondary treatment and $18 billion for new interceptors. As stated earlier, 47% of the facilities will require some treatment greater than secondary. The projected $21 billion are for the cost of the total facility to meet greater than secondary, not just that portion that results in greater than secondary effluent (i.e., tertiary treatment).

Technologies in the Construction Grants Program

A review of the present Construction Grants projects shows that a variety of technologies, both conventional, new and innovative, are being planned and constructed to meet the goals of PL 92-500. The following is a discussion of the factors that affect the selection of a process, particularly new processes, the projected treatment needs (1976 Needs Survey) and the results of a regional survey on the types of new systems that are being incorporated in the Construction Grants Program. The regional survey was in response to inquiries on the extent to which new and innovative technologies are being used in the Construction Grants Program. The general belief is that there is only limited use of new technology in the program.

For purposes of discussion, technologies have been divided into two categories; conventional and new. New technologies have been subdivided into proven and unproven.

Conventional Technologies - systems that have been widely employed for the treatment of municipal wastewaters and sludges for at least 20 years (trickling filters, activated sludge, lagoons, anaerobic digestion, vacuum filters, etc).

New Technologies

Proven - systems that have been evaluated on a full scale in the United States in the last 10 years (rotating bio-disc, oxygen activated sludge, air nitrification, lime chemical clarification, activated carbon, land application, oxidation ditches, etc.)
Unproven - systems that have not been evaluated on a full scale in the United States (sludge pyrolysis, co-pyrolysis, aquaculture, use of solar energy, energy recycling, alternative methods of disinfection, etc.)

In addition, there are a number of modifications to conventional systems that will be considered as new technologies. The author realizes that systems such as land treatment have been employed for many years in the United States. However, because these types of systems have not been widely used by sanitary engineers, they must overcome the same obstacles that face new technologies. For this reason, these technologies have been classified as "new" in this paper.

Factors that Effect the Process Selection.

There are two key factors which impact the implementation of a specific system in the Construction Grants Program -- cost-effectiveness and the level of confidence associated with the system. Successful implementation generally occurs when the system has been sufficiently developed and can economically compete with other systems in a cost-effective analysis.

Section 212 of PL 92-500 requires that applicants for construction grants provide sufficient information to demonstrate that the proposed system is the most cost-effective alternative for the degree of treatment required and disposal of waste solids. The cost-effectiveness analysis is conducted in the Step 1 facilities plan. The basis for the cost-effectiveness analysis is a comparison of the capital and O&M expenses for the project over a twenty year planning period. All grantees are required to consider land application and other forms of wastewater reuse as part of the cost-effective analysis. The Step 1 facilities plan, which includes the cost-effective analysis, is generally performed by the consulting engineer hired by the grantee, accepted by the municipality and approved by the State and EPA reviewing officials.

In order to aid the consulting engineer in performing the cost-effective analysis, EPA has published a series of documents. One such publication, "Alternative Waste Management Techniques for Best Practicable Waste Treatment" provides information on conventional and new systems for the treatment of wastewater and handling of residual sludges. It should be emphasized again that EPA does not dictate what processes should be used, only that the selected system must be cost-effective to meet the effluent quality required for the protection of the receiving stream. Cost-effectiveness is the key element in the selection of the treatment systems. Non-monetary factors (social and environmental) are accounted for descriptively in the cost-effectiveness analysis in order to determine their significance and impact. Both monetary and nonmonetary factors are considered in the final selection of the treatment system in the cost-effectiveness analysis.

New technologies must be developed sufficiently in order that their cost-effectiveness and reliability can be demonstrated. For conventional systems, this type of information and experience is available to the consulting engineer. In reality, a new technology system should only be selected over a conventional system if the system employing new technology is less costly and more environmentally acceptable. The procedure for obtaining the required information for new systems, particularly new-unproven systems, includes laboratory tests, bench tests, pilot plant studies and full scale performance evaluations. Full scale evaluation is hampered by the lack of research and development funds available for such a study. The EPA Construction Grant Regulations in section 35.908 provide that new processes, even those which have been demonstrated under less than full scale conditions, may be utilized in the construction of treatment plants. However, as a practical consideration, there may be a reluctance on the part of the potential designers, users and reviewers to utilize new technologies until confidence is gained for dissemination of data from successful full scale demonstrations.

In addition to the difficulties encountered by unproven new technologies, certain constraints exist for the implementation of new proven technologies that have been demonstrated on a full scale. The most obvious constraint is the preference for the selection of conventional processes by grantees and consultants because of their public acceptance and proven capability to meet effluent standards at a known cost. A recent EPA survey of the use of new technology, which included interviews with EPA Regional and State officials, has indicated six major reasons inhibiting the use of demonstrated new technology at the municipal level. First, familiarization and information transfer on demonstrated new technologies to gain public and consultant's acceptance, has been either slow or insufficient. Second, lack of adequate operating and cost data for new technology makes it difficult to develop an adequate cost-effectiveness alternatives comparison with conventional technology. Third, there is no incentive to consider and use (i.e., risk) new technology if the conventional technology appears to be adequate. Fourth, utilizing a new technology will often require expensive retraining of operating personnel. Fifth, many new and alternative technologies are simply not cost-effective and accordingly, cannot be funded by the grants program. And finally, EPA and State resources to review each facilities plan have been insufficient to fully evaluate the extent to which alternative technologies are considered.

The degree of familiarity that a State, EPA Regional Office, and most important, the consulting engineer has with a new technology, usually determines whether a new technology is selected. With over 8000 active grant projects (approximately 60% include treatment systems), a massive information transfer problem is created. A State, Region, or consultant often is not cognizant or familiar with developments of new technologies in other areas of the country. The problem of familiarity with a new technology is more acute for the smaller consulting firms. They are usually regional, have smaller staffs, and are involved in a fewer number of projects. Accordingly, smaller firms are less likely to know about or be familiar with demonstrated new technologies in other regions of the country.

This lack of information can also produce additional problems. The consultant or reviewing official may not be aware of difficulties that others have encountered with systems the consultant or reviewing official is presently recommending, designing, or approving.

An important constraint on implementation of sufficiently developed new technology is the preference for conventional systems by consultants. The survey corroborated other findings that consultants, needing to "serve" clients by ensuring projects achieve required effluent levels, will select conventional technologies. The fact is that the only product consultants can offer is their reputations.

Another important factor is that consultants may be **able** to realize greater profits by utilizing existing designs for conventional systems as opposed to redesigning for new technologies.

The fact that municipalities must bear the full financial burden of operating and maintaining treatment plants, as well as the 25% local share for capital costs in most cases, tends to add to the reliance on conventional technologies. The survey indicated that many municipalities, especially the smaller ones, can only afford low salary levels for plant operating personnel and low maintenance costs. Such a situation implies low skill level for operating personnel, which would preclude the effective use of some new technologies requiring a higher degree of sophistication to operate. In addition, municipalities are required to meet the effluent requirements of their permit. The threat of legal action as a result of a permit violation tends to compel municipalities to accept conventional systems.

Other constraints can be found at the State and Federal levels. EPA pressure to obligate Construction Grants funds quickly has resulted in greater implementation of conventional systems. This factor along with the lack of sufficient personnel at the State and Federal levels has encouraged consultants to recommend conventional systems which are more common to the individual reviewer.

Certain States have developed lists of approved processes and methods. Other states do not have such lists, but a proposal for funding of a large sized facility employing sufficiently demonstrated but new technologies would run the risk of not being favorably reviewed for grant funding.

An important factor which has influenced the utilization of some new technologies within the grants program has been the publicity or "salesmanship" which has been provided with the new system. Processes which have been developed and marketed by private industries such as pure oxygen activated sludge and rotating biological filters, have received wider acceptance than systems that required less private industry input, such as land treatment.

1976 Needs Survey.

The 1976 Needs Survey provides information on the number and types of treatment systems that will be enlarged, upgraded or constructed to meet the goals of PL 92-500. The Survey projects the treatment requirements and shows that by the year 1990, 34% of the population will require secondary treatment and 55% will require greater than secondary. Greater than secondary effluent does not necessarily mean that tertiary treatment will be required, only that an effluent greater than that defined by EPA as secondary effluent (30 mg/l of BOD and 30 mg/l of SS) will be necessary. Eleven percent of the population will remain unsewered.

Table 1 summarizes the types, number and flows of systems that are projected to treat wastewater in the United States. The category labeled "Other" includes systems which were not specifically labeled on the survey form. These systems may include both secondary and tertiary treatment systems.

As seen in Table 1, the majority of the wastewater will be treated by activated sludge systems. For the three specific types of secondary systems (activated sludge, trickling filter and oxidation ponds), activated sludge includes 55% of the facilities and nearly 80% of flow. Compared to oxidation ponds and activated sludge, the use of trickling filters is expected to decrease. Many trickling filter plants are being abandoned and replaced by other forms of secondary treatment because of the concern by engineers that trickling filter plants cannot consistently meet the secondary effluent requirements.

The fact that oxidation ponds are 9% of the flow and 34% of the facilities is understandable since over half of the treatment plants in the Nation have flows less than 0.1 MGD.

Tertiary treatment systems are also included in Table 1. As seen, filtration has the highest planned use. The use of the AWT systems (lime treatment, breakpoint chlorination, ion exchange) is very limited. The number of land treatment systems is considerably lower than those rojected by other data which will be discussed later.

The information on tertiary treatment can be compared with Needs Survey data on treatment plants that reported "greater than secondary required." The survey showed that 9036 facilities will require an effluent quality greater than that defined by EPA as secondary effluent (30 mg/l of BOD and 30 mg/l of SS). Assuming 19,000 treatment plants, it can then be projected that 47% of treatment plants will require "greater than secondary." Population projections show that the 47% facilities correspond to 55% of the projected sewered population. It is important to point out again that "greater than secondary" does not necessarily mean that tertiary treatment will be required. This fact is shown in Table 2. The 9036 facilities that responded "yes" to the question of "greater than secondary" were sampled to determine their projected effluent quality (Table 2). Effluent quality data from all responding plants was not provided on the survey forms. However, if as a minimum, additional BOD removals were required by all responding plants, the data in Table 2 would represent 85% of those plants where greater than secondary would be required.

While 7704 facilities must provide an effluent with less than 30 mg/l of BOD, only 6271 facilities must do better than 30 mg/l of suspended solids (SS). These results are consistent with many permit limitations, based on water quality standards, where BOD requirements are generally lower than SS requirements. Assuming that filtration would be required to attain a BOD of less than 20 mg/l, then the 4927 facilities in Table 2 are consistent with 4993 filtration facilities in Table 1. Apparently, facilities expect to attain less than 5 mg/l of BOD by reducing the suspended solids to less than 10 mg/l.

Facilities requiring phosphorus removal are also presented in Table 2. Phosphorus removal was assumed to be required when the effluent limitation was less than 2 mg/l of phosphorus. As seen, only 56 plants require phosphorus concentrations less than 0.5 mg/l and 180 plants require phosphorus concentrations between 0.5 and 0.9 mg/l. This data is consistent with the two-stage lime projects (70) listed in Table 1, since two-stage lime would generally be applied when the effluent quality was less than 0.5 mg/l of phosphorus. The majority of the plants that need phosphorus removal (83%) fall in the 1-2 mg/l range which can be produced by chemical addition to the secondary system. Chemical addition to the secondary system will also result in BOD and SS concentrations less than 30 mg/l. At the present time, chemical addition to primary or secondary systems is used at approximately 400 facilities.

Nitrogen requirements are shown by the ammonia and total nitrogen concentrations in Table 2. The data show that the majority of the plants will require ammonia conversion to nitrate and not total nitrogen removal. Table 1 reported 653 separate biological nitrification systems while Table 2 lists 2666 projects requiring ammonia removal. The addition of ion exchange, breakpoint chlorination or projected ammonia stripping cannot approach the number of systems reported in Table 2. It would appear that the majority of the nitrification systems are single-stage systems such as oxidation ditches or extended aeration plants. The need for high removals of total nitrogen (less than 5 mg/l) is small with approximately 200 facilities. By comparing these results with facilities which must provide an effluent phosphorus concentration of less than 1 mg/l and suspended solids of less than 5 mg/l, it can be seen that the highly sophisticated AWT systems are probably limited to 200 facilities. As mentioned earlier, many facilities expect to attain a BOD less than 5 mg/l by reducing the suspended solids to less than 10 mg/l. However, as seen in Table 1, the use of activated carbon (BOD less than 5 mg/l) is projected for 295 facilities.

Needs Survey information on sludge processes is presented in Table 3. As seen, the category labeled "other" contains more projects than any other single process. It is assumed that most of these projects include some type of land disposal of sludge. The increased emphasis on reducing energy usage has affected the use of incineration. However, the incineration industry has responded by the development of alternative systems to decrease energy consumption, which will be discussed later.

EPA Survey on New Technologies.

Congressional committee hearings on science and technology raised questions concerning the utilization of new technologies in the EPA Construction Grants Program. As a result, a survey was initiated in November 1976 to determine what new technologies were being used and what factors influenced their implementation. Each of the ten EPA Regional Offices was supplied with forms containing new wastewater and sludge treatment systems (Attachment 1). Forms were completed for projects (completed Step 1, Step 2, and Step 3) which included a specified system listed on the form. In addition, systems which were considered new in the opinion of the EPA official but were not on the list were included. EPA headquarters personnel visited each Regional Office, completed the forms and discussed the results with the EPA regional staff. In addition, EPA headquarters staff had the opportunity to interview some State representatives.

The data collected from the survey has been summarized in Tables 4 through 8. In addition to the presented information, the survey found a number of systems being employed that are not included in the tables. These included, screens to replace primary settlers, use of solar energy, digester gas utilization, heat recovery, automatic computer control systems, aquaculture, electric power generation in a gravity sewer and alternative disinfection systems such as ozone. Over 1000 projects were identified in the survey as employing new technology. Because of time constraints and work loads, not all projects that included new technologies were reviewed and summarized. Emphasis was placed on providing information on different types of systems and as a result, all projects which included systems with higher usage were not reviewed. This was the case with systems such as rotating biological contactors (RBC's) and oxidation ditches whose use has been widespread.

Systems that are used to achieve secondary effluent are included in Table 3. Land application, which achieves greater than secondary effluent was also included in Table 3. The data shows a widespread use of oxygen activated sludge, RBC's and oxidation ditches for secondary treatment.

All of these types of systems are marketed by private firms. Oxidation ditches and RBC's are the two systems whose use has increased rapidly in the United States. A draft report by William F. Ettlich of Culp, Wesner and Culp titled "A Study of Oxidation Ditch Plants and Technology" lists 544 installations in the United States. Mr. Ettlich lists 5 reasons for the increased use of oxidation ditches.

1. Construction costs equal to or less than competitive treatment processes.
2. Plants require a minimum of mechanical equipment.
3. Plants appear to perform reasonably well even with minimum operator attention, primarily due to conservative design.
4. Waste sludge is relatively nuisance free and is readily disposed of at most plants.
5. Plants generally do not generate odors even under poor operating conditions.

RBC's have the capability to be retrofitted into an existing facility for upgrading or enlarging. These units also lend themselves toward modular design. Pure oxygen activated sludge systems have been developed and promoted by several large companies. These systems have been marketed based on retrofitting to increase capacity or capability of handling high organic strength waste.

The information on land treatment systems in Table 3 shows that far more land application systems are being employed than is shown in the land treatment data in Table 1. Table 1 projects 265 land treatment systems by 1990 while Table 3 shows 242 projects at the completed Step 1, Step 2 or Step 3 stages. The data in Table 3 was a sampling of regional projects and did not include all land application projects. As seen, land application is used more in the West where water supplies are limited. Activated Biological Filters were not found in the northeast section of the United States.

Projects for the nitrification of municipal wastewater are presented in Table 5. As seen, separate stage air systems and RBC's were found to be the most widely used. Although Table 5 only lists 11 oxidation ditch systems designed for nitrification, many oxidation ditch systems are expected to produce a nitrified effluent. The same is true of many extended aeration systems which could be listed under "combined air."

Table 6 lists various tertiary treatment systems. This information is consistent with the data shown in Table 1. As seen, filtration is the highest used form of tertiary treatment. Microscreens are also being employed to improve effluent quality in small facilities. The data shows that the use of the AWT systems (lime clarification, NH_3 stripping, activated carbon, breakpoint chlorination and ion exchange) has not been widespread. This data is also supported by the Needs Survey Data in Table 1. The effluent quality data in Table 2 shows that the need for the AWT systems (high levels of phosphorus, SS, BOD, and total nitrogen) apparently does not exist.

These systems were developed in the late 60's and early 70's within the Advanced Waste Treatment Research Program. The major objective of this program was to develop treatment processes for maximum removal of contaminants and repeated reuse of the Nation's waters. The level of treatment produced by these processes is not necessary to produce the effluent qualities now required. Because of the additional expense to build, operate and maintain these types of systems, the application of these systems will no doubt remain limited.

Similar comments can be made concerning the use of the independent physical/chemical systems for wastewater treatment. Data on these systems is presented in Table 7. A number of factors can be attributed to the limited use of this type of treatment.

1. Effluent standards do not require the levels of treatment produced from these systems.
2. Energy usage, because of the recalcination of lime and regeneration of activated carbon, is extremely high.
3. Systems for nitrogen removal have not been demonstrated on a full scale.
4. Systems are generally not cost-effective as compared to land application when stringent effluent qualities are required.

The draft EPA report "Energy Conservation in Municipal Wastewater Treatment" by Culp, Wesner and Culp Consulting Engineers, compared the energy usage for a conventional treatment system and an IPC system to produce secondary effluent at a flow of 30 MGD. The conventional system which included activated sludge with incineration of the waste solids required $31,600 \times 10^6$ BTU/yr and $10,665 \times 10^3$ KWH/yr of total energy. The IPC system which included incineration of the chemical sludge and regeneration of activated carbon required $275,000 \times 10^6$ BTU/yr and $10,777 \times 10^3$ KWH/yr of total energy. The escalating costs of fuel in addition to the availability, which must be considered in the cost-effective analysis, is a major factor that will limit the use of these systems.

The information on solids processing projects is included in Table 8. Compared to wastewater treatment systems, this information is relatively small. Heat treatment systems appear to be the most widely used. Of all the systems, including wastewater treatment, heat treatment systems were frequently identified as not operating properly.

Concern for odors and possible health effects in projects employing land disposal of sludge has led to the use of chlorine oxidation and lime stabilization. The possible production of chlorinated hydrocarbons during chlorination of organic sludges has resulted in additional studies on this system. Sludge composting has become increasingly popular since composting raw sludge without excessive odors has been demonstrated.

Energy consumption for solids processing has had major impact on systems presently being considered. Disposal of sludge can be energy intensive if fuels are required. Sludge is also the only byproduct which offers the potential for energy recovery. Methane gas, the byproduct of anaerobic digestion, is being used for power generation and direct coupled combustion engines. The energy report by CWC shows that methane gas from anaerobic digestors could provide 80% of the electrical energy requirements of the primary and secondary treatment systems.

The volatile fraction of the waste solids are a potential source of energy. To reduce fossil fuel requirements for incineration, increased attention has been given to filter presses to reduce the liquid fraction for incineration without auxiliary fossil fuels. Data from Envirotech Corporation shows that 22 facilities are being constructed to employ heat recycling to further decrease the water content prior to incineration.

Pyrolysis of sewage sludge, because of its potentially lower fuel inputs and lower air emissions, is currently being evaluated to handle the sludge for the New York/New Jersey area.

Co-incineration with solid waste is being used to reduce fossil fuels. Co-pyrolysis of sewage sludge and solid waste is being planned for the Contra Costa wastewater treatment plant. Studies have shown that sufficient energy can be developed for co-pyrolysis to supply the average demand for a 30 MGD AWT facility.

SUMMARY

Treatment systems, employing new technologies and particularly new unproven technologies, must overcome a series of obstacles before they are accepted and utilized by consulting engineers and municipalities in the Construction Grants Program. The use of these systems was found to be a function of their familiarity and acceptance by engineers, adequate cost information, and needs to attain desired effluent qualities. Familiarity and acceptance can be accelerated by an active sales program by private industry.

The criticism that new technologies are not being employed in the Construction Grants Program is not supported by our recent surveys. At the present time, approximately 2,500 Step 2 and 3 grants include treatment plants. The EPA survey of regional files showed that 697 projects, or 28%, are using new technologies. The percentage is the minimum amount, since not all new technology projects were identified in the survey.

The use of Advanced Waste Treatment plants to attain very high removals of BOD, suspended solids, phosphorus and total nitrogen appears to be limited to 200 of the 19,000 facilities that require upgrading, enlarging or construction. Projected effluent limitations and high costs of such facilities limit their application.

The amount of land application projects will be higher than previously projected. Limited water supplies for irrigation in various sections of the United States will probably have the greatest impact on the utilization of this technology. The use of land treatment only in those situations where very high removals of pollutants are required, based on projected effluent limitations, would be limited to approximately 200 facilities. Our recent survey showed approximately 350 land treatment systems out of 2,500 treatment plant projects.

Compared to wastewater treatment technologies, utilization of new sludge handling and disposal techniques in the program is lagging. This apparent delay is a result of the lack of new systems to be demonstrated or their unsuccessful demonstration. Successful demonstration of energy savings and safe ultimate disposal techniques will aid in correcting this situation.

REFERENCES

Pound, C.E., Crites, R.W., and Smith, R.G., "Cost-Effective Comparison of Land Application and Advanced Wastewater Treatment," EPA 430/9-75-016; Office of Water Program Operations, Environmental Protection Agency, November 1975.

TABLE 1

1976 NEEDS SURVEY TREATMENT PROCESSES

TYPE OF SYSTEM	IN USE	UNDER CONSTRUCTION	PLANNED	TOTAL	
	NUMBER OF PROJECTS	NUMBER OF PROJECTS	NUMBER OF PROJECTS	NUMBER OF PROJECTS	FLOW m^3/DAY X 10^3
ACTIVATED SLUDGE	3,550	559	5,847	9,956	100,738
TRICKLING FILTER	1,727	42	206	1,975	13,758
OXIDATION PONDS	3,504	214	2,527	6,245	11,507
LAND TREATMENT	51	10	204	265	1,652
OTHER	1,883	261	4,001	6,145	64,076

TABLE 1 (CONTINUED)

1976 NEEDS SURVEY TREATMENT PROCESSES

TYPE OF SYSTEM	IN USE	UNDER CONSTRUCTION	PLANNED	TOTAL	
	NUMBER OF PROJECTS	NUMBER OF PROJECTS	NUMBER OF PROJECTS	NUMBER OF PROJECTS	FLOW m^3/DAY X 10^3
FILTRATION	535	207	4,251	4,993	48,383
ACTIVATED CARBON	13	13	269	295	3,400
TWO STAGE LIME	15	7	48	70	1,609
BIOLOGICAL NITRIFICATION	32	37	584	653	15,193
BIOLOGICAL DENITRIFICATION	11	7	120	138	3,596
ION EXCHANGE	0	3	3	6	
BREAKPOINT CHLORINATION	25		82	107	628

TABLE 2

1976 NEEDS SURVEY
TREATMENT PLANS AND EFFLUENT QUALITY

BOD EFFLUENT		SS EFFLUENT		TOTAL P		TOTAL N		AMMONIA-N	
mg/1	# OF PLANTS	mg/1	# OF PLANTS	mg/1	# OF PLANTS	mg/1	# OF PLANTS	mg/1	# OF PLANTS
<5	796	<5	122	<0.5	56	<1	3	<1	89
5–9	1134	5–9	1559	0.5–0.9	180	1–4.9	198	1–1.9	1098
10–19	2997	10–19	2497	1–2	1186	5–9.9	110	2–5	1479
20–29	2777	20–29	2093			10–15	77		
TOTAL	7704		6271		1422		388		2666

TABLE 3

SLUDGE PROCESSES 1976 NEEDS SURVEY
(Number of Projects)

	IN USE	UNDER CONSTRUCTION	PROJECTED	TOTAL
ANAEROBIC DIGESTION	4803	296	3377	8476
HEAT TREATMENT	165	50	124	339
AIR DRYING	5272	286	3910	9468
DEWATERING	1147	241	1701	3089
INCINERATION	1316	105	913	2334
LANDFILL	1724	127	1784	3635
LAND SPREADING	584	36	322	942
OTHERS	5939	396	4005	10,340

TABLE 4

BIOLOGICAL SECONDARY TREATMENT
(Number of Projects)

	EPA REGIONS										
TREATMENT PROCESS	I	II	III	IV	V	VI	VII	VIII	IX	X	TOTAL
OXYGEN AERATION	6	8	11	16	10	5	5	2	9	9	81
RBC	6	3	18	9	33	1	24	6	4	12	121
PLASTIC MEDIA			5	2	7	3	5		1	1	24
ACT. BIOLOGICAL FILTER				2	2	3	5	4	4	15	35
OXIDATION DITCH	18	14	6	11	31	13	36	10	15	22	176
LAND APPLICATION*	10	1	1	25	14	31	7	17	92	44	242

*ACHIEVES EFFLUENT GREATER THAN SECONDARY

TABLE 5

NITRIFICATION
(Number of Projects)

TREATMENT PROCESS	EPA REGIONS I	II	III	IV	V	VI	VII	VIII	IX	X	TOTAL
SEPARATE STAGE AIR	2	5	11	7	10	5	3	5	10	2	60
COMBINED AIR	3	2		3			1	1			10
SEPARATE STAGE O_2		4	1	6	2	1	2				16
COMBINED O_2		1									1
RBC	1	6	7	5	13		6			1	39
DITCH		6	4	1							11
ACT. BIOLOGICAL FILTER				2			1	1		1	5

TABLE 6

TERTIARY TREATMENT
(Number of Projects)

TREATMENT PROCESS	EPA REGIONS I	II	III	IV	V	VI	VII	VIII	IX	X	TOTAL
LIME CLARIFICATION	3	2	8	3	1	3		3	5	5	33
ALUM CLARIFICATION								2	1	3	6
FILTRATION	5	6	17	9	17	14	14	9	14	15	120
MICROSCREENS			10	2	19				1		32
AMMONIA STRIPPING	1					1			3	1	6
POWDERED CARBON	1			1	1						3
GRANULAR CARBON	1	2	8			2	1	6	4	2	26
B.P. CHLORINATION	1		3					1	1		6
ION EXCHANGE			2			1			1		4

TABLE 7

INDEPENDENT PHYSICAL/CHEMICAL TREATMENT

(Number of Projects)

TREATMENT PROCESS	EPA REGIONS I	II	III	IV	V	VI	VII	VIII	IX	X	TOTAL
LIME CLARIFICATION	2	3	1		1	1			3		11
ALUM CLARIFICATION		2			1				1		4
FILTRATION	2	5	1		2	1			2		13
GRANULAR CARBON	1	5	1		2	1			2		12
B.P. CHLORINATION		2									2
ION EXCHANGE					1						1

TABLE 8

SOLIDS PROCESSING

(Number of Projects)

TREATMENT PROCESS	EPA REGIONS I	II	III	IV	V	VI	VII	VIII	IX	X	TOTAL
SLUDGE COMPOSTING	3	1	2			1		1	1		9
CHLORINE OXIDATION	3		3								6
HEAT TREATMENT	4	4	5	1	10	2	2	3	1	3	35
LIME STABILIZATION	1	1	2		1	1	1			1	8
FILTER PRESS	4	2	2		4						12
BELT FILTER	1	1	1	1	2	1				2	9
PYROLYSIS		1								1	2
CO-PYROLYSIS		1	1								2
CO-INCINERATION	2	2	3		2	2					11
FERTILIZER			2		5	2	4	1	1		15

ATTACHMENT I

Project No:______________________ EPA Regions:____________________________

Grant Applicant:______________________ Name of Preparer:______________________

Location:______________________________ Telephone No:__________________________

*Check one: PL - 92-500 Project Step 2 Grant Awarded ______
PL - 92-500 Project Step 3 Grant Awarded ______
Old Law Project ______ Step 1 Study Completed ______

*Date of award (bid)______________________________________
*Design average flow (MGD)________________________________

WASTEWATER:
OXYGEN ACTIVATED SLUDGE ______
SPECIFY PROCESS____________________
OTHER OXYGEN SYSTEM ______ (specify) __________
ROTATING BIOLOGICAL CONTACTOR (Disc) ______
  for BOD REMOVAL ______
  for NITRIFICATION ______
OXIDATION DITCH ______
ACTIVATED BIO FILTER ______
PLASTIC MEDIA TRICKLING FILTER ______
LAGOON (POND) UPGRADING ______
  WITH ROCK FILTERS ______
  INTERMITTENT SAND FILTER ______
  SUBMERGED SAND FILTER ______
  OTHER ______ (SPECIFY)____________________
LAND APPLICATION (TREATMENT) ______
  CROP IRRIGATION
  (or slow rate systems) ______
  RAPID INFILTRATION ______
  OVERLAND FLOW ______
CARBON ADSORPTION ______
  GRANULAR ______
  POWDERED ______
  NEW FILTRATION TECHNIQUES ______
  (NOT RAPID SAND FILTERS)
  MULTI OR DUAL MEDIA ______
  SPECIFY________________________
CARBON REGENERATION ______
LIME TREATMENT ______
  WITH BIOLOGICAL TREATMENT ______

  AFTER BIOLOGICAL TREATMENT (tertiary) ______
  BEFORE NITRIFICATION ______
  PHYSICAL - CHEMICAL
  (no biological treatment) ______
RECARBONATION ______
ION EXCHANGE ______
REVERSE OSMOSIS ______
AMMONIA STRIPPING ______
TUBE SETTLERS ______
MICRO SCREENS ______
NITRIFICATION ______
  SUSPENDED GROWTH (TANKS) ______
  ATTACHED GROWTH (FILTERS) ______
  WITH AIR ______ WITH OXYGEN ______
    FLUDIZED BEDS ______
DENITRIFICATION ______
  SUSPENDED GROWTH (TANKS) ______
  ATTACHED GROWTH (FILTERS) ______
COMBINED NITRIFICATION-
DENITRIFICATION ______
WASTEWATER REUSE ______
  GROUND WATER RECHARGE ______
  RECREATIONAL REUSE ______
  INDUSTRIAL REUSE ______
  OTHER (SPECIFY)______________________

WASTEWATER:
UTILIZATION OF SOLAR ENERGY
(COLLECTORS OR CELLS) ______
NEW DISINFECTION TECHNOLOGY ______
  OZONE ______
  ULTRA VIOLET LIGHT ______
  BROMINE CHLORIDE ______
  OTHER ______
    SPECIFY________________
DECHLORINATION ______
  SULFUR DIOXIDE ______
  OTHER ______
  SPECIFY______________________
STORM AND COMBINED SEWER ______
OVERFLOW TREATMENT ______
  SPECIFY TYPE________________
NON-SEWERED TREATMENT ______
  SPECIFY TYPE________________
OTHER INNOVATIVE PROCESSES ______
  SPECIFY______________________

SLUDGE:
CO - INCINERATION ______
CO - PYROLYSIS ______ PYROLYSIS ______
REGIONAL TREATMENT OF SEPTAGE
TANK PUMPINGS ______
SLUDGE COMPOSTING ______
LIME CONDITIONING ______
UTILIZATION OF INCINERATOR ASH FOR
SLUDGE CONDITIONING ______
CHEMICAL FIXATION (STABILIZATION)
OF SLUDGE ______
  SPECIFY PROCESS________________
COMMERCIAL SOIL CONDITIONER/
FERTILIZER PRODUCTS ______
DIGESTER GAS DRIVER INTERNAL
COMBUSTION ENGINES ______
NEW SLUDGE DEWATERING TECHNIQUE ______
  HEAT TREATMENT ______
  FILTER PRESS ______
  BELT FILTER ______
  UTILIZATION OF WASTE HEAT ______
  SPECIFY________________________
OTHER INNOVATIVE PROCESSES ______
  SPECIFY________________________

  ____

COMPUTERIZED PROCESS CONTROL ______

DISCUSSION

OTHER INFORMATION AVAILABLE ON THE TECHNOLOGIES (PROCESSES) CHECKED ABOVE (VARIATIONS, PECULIAR CIRCUMSTANCES, ETC.)

UTILIZATION OF A FEDERAL ACT (PL 92-500) IN COORDINATION WITH GEORGIA'S WATER QUALITY CONTROL PROGRAM

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The 1972 Federal Water Pollution Control Act (PL 92-500) instantly impacted the various State water pollution control programs in the United States. At first the sudden shift of significant statutory authorities from a State function to a Federal function created considerable controversy and confusion; however, as time passed, the Federal and State roles were clarified and progress is being made toward abatement of water pollution throughout the Nation. The State of Georgia had an active and effective water pollution control program which had been initiated in 1964. Through the additional financial support to the State's program provided by Titles I and II of PL 92-500, Georgia improved its water quality monitoring network and data processing, including computer modelling. In addition, using the additional legal authorities established under Titles III and IV of PL 92-500, Georgia strengthened its enforcement program through utilization of the permit program and more stringent levels of treatment. By 1977 sufficient progress has been made by Georgia's water quality control program, in coordination with PL 92-500, to enable the State to meet water quality standards for almost all rivers, lakes, and estuaries.

The enactment of the 1972 Federal Water Pollution Control Act (PL 92-500) by the U. S. Congress over a Presidential veto established a far-reaching complex national commitment to eliminate water pollution problems in the United States. Instantaneously the water pollution control program was politicized to an unprecedented degree. Prior to PL 92-500, the individual States had used various approaches to regulate the discharge of pollutants from industrial and governmental operations within their own boundaries; however, this role was superseded by the U. S. Environmental Protection Agency(EPA) under PL 92-500. Simultaneously the Act implemented another precedent for the program - the concept of contractual obligation and the authorization for EPA to proceed with an eighteen billion dollar construction grant program. This establishment of a strong Federal role and authorization of the $18 billion for 75 percent construction grants assured a high level of interest and involvement by politicians from the Federal, State, and local government levels.

No single piece of environmental legislation in the United States' history has provoked as much interest and controversy as PL 92-500. For the past four years, the various professional, technical and trade journals and publications have been filled with articles on this subject. In addition, thousands of governmental publications and newspaper articles have concentrated on the water pollution control program. During this period, numerous conferences and seminars have been conducted on the subject. A significant portion of this attention to the program has been in a negative tone with sharp criticism leveled at the Federal EPA for its administration of the program. Much of this criticism has been related to the delays encountered in the construction grants program which have delayed the abatement of pollution from publicly owned wastewater treatment works. Significant problems have occurred in the development and implementation of effluent limitations for various industrial dischargers which could be considered reasonable and acceptable to the industrial community. The overly optimistic deadlines in the Act for certain accomplishments have been criticized.

Section 315 of the Act established the National Commission on Water Quality and required a three-year study. Fifteen million dollars were authorized for the Commission to investigate and study the technological aspects as well as the economic, social, and environmental effects of the far-reaching objectives of the Act. To a great extent, the Commission's study and report resulted in additional controversy for the program.

Since early 1974, numerous Congressional committees and subcommittees have conducted investigations and hearings in an effort to define problems related to this program and to consider possible corrective actions. In late 1976, the two houses of Congress failed to agree on any amendments to PL 92-500; therefore, no significant amendments have been made to the Act. The debates and controversies surrounding PL 92-500 have continued into 1977.

A purpose of this paper is to present some positive aspects and accomplishments during this controversial period, specifically related to the State of Georgia. Although some of the deadlines and goals of PL 92-500 have not been met, substantial progress continues to be made toward abatement of water pollution.

To place the impact of PL 92-500 into proper prospective for the Georgia water quality control program, a few facts regarding Georgia and the State's program prior to 1972 must be presented. Georgia, one of the original thirteen colonies, is located on the east coast in the southeastern portion of the United States. With an area of approximately 152,810 square kilometers (59,000 square miles), Georgia is the largest State east of the Mississippi River. The State has a population of about five million with one third of the population located in the Atlanta area. No rivers flow into Georgia from other States; however, numerous streams originate in the State and flow in a southerly direction. Some of the rivers flow into the Atlantic Ocean while others flow into Florida or Alabama and thence to the Gulf of Mexico. Most of the State's population live in the northern half of the State where the streams are small and water quality problems are more severe because of these population pressures. The State does receive an annual rainfall of about 1219 mm (48 inches).

Prior to 1964, Georgia had made limited efforts to control water pollution; however, following enactment of the 1964 Georgia Water Quality Control Act an effective program was implemented. Beginning in early 1965, a technical staff of engineers, chemists, and biologists located and inventoried wastewater dischargers in the State. Ambient water quality standards and stream use classifications were established for the State's streams in 1966 and 1967. Wastewater dischargers were placed on schedules to install treatment facilities. In general, the local governments and industries were required to provide 85 percent reduction of organic waste loads as measured by biochemical oxygen demand (BOD) and to eliminate the discharge of toxic pollutants. The schedules established the end of 1972 as the target date for installation of these treatment facilities.

Less than 10 of the 440 industries discharging significant quantities of wastewater directly to streams in the State failed to meet the 1972 deadline. In general, the installed treatment facilities provided the level of treatment, five years in advance, which was required by July 1, 1977 in PL 92-500. With local governments, the progress was far less successful. There were 480 publicly owned wastewater treatment works in Georgia which serve a population of 2.9 million or about 60 percent of the State's population. These publicly owned wastewater treatment works have a designed capacity of 2.4×10^6 m^3/d (637 mgd) and are currently treating approximately 1.5×10^6 m^3/d (400 mgd) of wastewater.

Financial and management problems hampered the efforts to correct water pollution problems in local governments. Prior to the enactment of PL 92-500, there had been limited financial assistance from the Federal government.

In retrospect the enactment of PL 92-500 was timely for the State of Georgia. Industrial dischargers had installed modern water pollution control facilities, but local governments were encountering financial difficulties. Therefore, the authorization of 75 percent Federal grants by PL 92-500 would enable local governments to proceed with programs to install needed facilities. Also, the Georgia program previously had concentrated on correcting water pollution problems related to point sources, whereas PL 92-500 enabled the State to place emphasis on developing a broader program which would address all aspects of water quality management.

Georgia's water pollution control program is administered by the Environmental Protection Division of the Department of Natural Resources. Through the assistance of Governor Jimmy Carter and the Georgia Legislature, the Georgia Water Quality Control Act was amended in 1973 to make it consistent with PL 92-500.

Utilizing Federal funds provided to the State under Section 106 of the Federal Act, Georgia proceeded to modify its water pollution control program in 1973 in order to implement the various elements of PL 92-500. In accordance with Title III of the Act significant changes were made to the State program. A continuing planning process for water quality management was established as required by Section 303(e) of PL 92-500. The planning process enabled the State to establish for each river basin a priority ranking of needed construction or improvements of wastewater treatment works. Also, a total maximum daily load was determined for pollutants allowed to be discharged to each stream segment to assure compliance with water quality standards.

An expanded ambient water quality monitoring and surveillance program was developed to better identify the quality of the State's streams. Considerable improvements were made in the State's ability to compile and evaluate water quality data. Through the use of computers, the information was stored and evaluated, subsequently much of the information was used to develop computer models to simulate various stream reaches and the impact of pollutional loads on the streams.

The State's improved water quality monitoring and data evaluation enabled various stream reaches to be identified as "water quality limited" or "effluent limited". If the pollutional loads from waste dischargers, assuming installation of best practicable treatment (BPT), along a particular stream were not predicted to sufficiently protect water quality to meet the water quality standards for that reach, the stream was identified as "water quality limited" and more stringent levels of treatment were required. If the computer model predicted that installation of BPT would allow water standards to be met, the stream was identified as "effluent limited" and BPT was the required level of treatment. This more scientific approach to the establishment of required levels of treatment was an improvement of the State's program and a positive impact of PL 92-500.

As previously discussed, the provision of 75 percent Federal construction grants by PL 92-500 was an essential and positive element of the Act. Requirements for the preparation of a "facility plan" under Section 201 that would assure proper identification of needed improvements and the evaluation of alternatives for the provision of the most cost-effective system to provide these improvements was a major positive aspect of the program. For the first time, local governments had assistance and support from Federal and State governmental agencies in assuring that their technical consultants developed a cost-effective solution. Another major aspect of the 201 facility plan has been the requirement for an evaluation of inflow-infiltration problems for publicly owned systems. These studies have identified repairs which can be made to the sewage collection system to reduce flows which in turn reduce construction, operation, and maintenance costs for treatment works.

The Georgia Environmental Protection Division has coordinated the State developed water quality plans under Section 303(e) of the Act with the 201 facility plans developed by local governments to identify the level of treatment required at publicly owned systems to protect the State's streams. Since more than 99 percent of the sewage from publicly owned systems in Georgia is presently receiving biological treatment, the emphasis on the program for the future is to upgrade and improve existing systems and install advanced waste treatment where needed to protect downstream water uses. Downstream water uses on many of Georgia's relatively small streams include water supply for other communities and recreational lakes which collect and store many of the pollutants from upstream discharges. Therefore, advanced waste treatment systems consisting of 95 percent reduction of BOD, reduction of phosphorus to 1.0 mg/l, reduction of ammonia to 2.0 mg/l, the provision of 6.0 mg/l of dissolved oxygen (DO) in the effluent, and disinfection of the effluent are being required for several systems in the northern half of Georgia. Present studies do not verify that these levels of treatment will correct all water quality problems; however, it is predicted that with these levels of treatment, present downstream water uses can continue with reasonable assurance that the health and welfare of the public are protected.

A significant program authorized in Section 109 of PL 92-500 is one for training persons in the operation and maintenance of treatment works. In Georgia, a full-time training school for operators has been established in cooperation with the State Department of Education. Three full-time personnel conduct the training school for operators of treatment works. A wastewater treatment facility is being constructed for training purposes at the school. Also, an on-the-job training program has been effectively functioning for several years.

One of the major positive elements of PL 92-500 is the National Pollutant Discharge Elimination System (NPDES) permit program established under Section 402 of the Act. Through the requirements that each discharger obtain a permit, the NPDES permit program has implemented the specific effluent requirements established for various industrial and publicly owned systems. Georgia was approved by EPA to administer the NPDES permit program in early 1974 and has used it effectively to establish schedules and conditions for further improvements needed by certain dischargers.

The NPDES permit program has enabled several major accomplishments to be made. It has been an effective enforcement tool and the basis for substantial civil penalties throughout the United States. Implementation of the NPDES permit program has brought equity to the enforcement efforts across the country. If a State has been unwilling to administer a strong effective program, then EPA has continued to conduct the NPDES permit program. The maintenance of a strong, consistent program has assured various industries that have installed modern water pollution control facilities in Georgia that their competitors will do likewise in other parts of the country. This assurance has been extremely helpful to the States with effective ongoing water pollution control programs. Another major advantage of this uniformity is the fact that an industry proposing to locate a new plant must install minimum treatment levels in any one state compared to another state.

Another important and positive aspect of the water pollution control program under PL 92-500 has been the emphasis on public participation as required under Section 101(e) of the Act. Understanding and involvement of the Public are essential to the success of the program. In order to maintain strong support and commitment to the water pollution control program with the long-term substantial levels of funding required to accomplish the goals of the program, the citizens of Georgia and the Nation must understand and actively advocate it.

Georgia, along with all the other States, is now underway with the development of a statewide plan as required under Section 208 of the Act. The 208 plan is scheduled to be complete by November 1978. As required by the Act and rules promulgated by EPA, the 208 plan will provide Georgia with a comprehensive water quality management plan for the future. The 208 plan will place emphasis on non-point source pollution as well as point source pollution. Since a program has already been developed under Section 303(e) for point sources, a major effort of the 208 program in Georgia will be to identify the source and scope of non-point pollutants. Then a program will be developed to reduce and control non-point sources in conjunction with point sources. It is anticipated that completion of the 208 plan with periodic updating will result in a management tool for the future to assure adequate protection and management of the State's water resources.

The 1977 status of the water quality management program in Georgia reflects efforts under the State's program before and after PL 92-500. Industry in Georgia will meet the 1977 requirements of the Act. Several publicly owned systems will not meet the 1977 requirements primarily because the Federal construction grant funds were not provided as authorized by PL 92-500.

Since enactment of PL 92-500, almost $250 million of Federal construction grant funds have been obligated for improvements to publicly owned treatment works in Georgia. Many of these projects are now complete or under construction. The results of this program and the State's earlier efforts have greatly improved water quality in the State. In addition, the construction grants program has provided a positive impact on the economy by providing jobs and an economic stimulus during a recession period of the economy.

As of 1977, four and one-half years following enactment of PL 92-500, there is considerably less water pollution in the State of Georgia. Of the State's 32,180 kilometers (20,000 miles) of streams, less than 1609 kilometers (1,000 miles) have significant water pollution problems that result in violation of water quality standards. Perhaps of more importance is the fact that to a great extent PL 92-500 has enabled the State to enter the third century of our Nation better equipped, legally and technically, to manage effectively the water resources of the people.

FEDERAL-STATE-REGIONAL DEVELOPMENT OF A WASTEWATER MANAGEMENT PLAN AND A WATER SUPPLY PLAN

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Atlanta, Georgia is a major population center in the southeastern United States. Since the region is located on one of the major drainage divides of the United States, the water resources adjacent to the region are limited which results in serious water supply and water quality control problems. In order to meet the needs of the future, the region must plan and implement a sound water resources management program. The Federal government, the State government, and the Atlanta Regional Commission (representing 46 local governments) have developed plans to meet the needs of the region to the year 2000. Existing systems have been evaluated and stream studies have been conducted. Twenty-four biological wastewater treatment works with a total design capacity of 12.5 m^3/s (286 mgd) and the impact of the discharges on the three rivers draining the region have been evaluated. Using the results of these studies and the population prediction for the year 2000, it has been determined that water quality standards can be met and the necessary water supply provided until the year 2000. In developing the wastewater management plan, which required advanced waste treatment for all of the region, major emphasis was placed on protecting downstream water uses, particularly community water supply intakes and recreational lakes. Conservation of water and water reuse are additional considerations that must be fully evaluated in the next few years. If new technology is not developed and if water conservation is not practiced, the lack of water may well be the controlling growth factor for the future of the region.

THE PLANS

During the past three years comprehensive and cooperative efforts have been made to develop a wastewater management plan and a water supply plan for the metropolitan area of Atlanta, Georgia. An in-depth study has been conducted by the U. S. Environmental Protection Agency (EPA), the U. S. Army Corp of Engineers, the Georgia Environmental Protection Division, and the Atlanta Regional Commission. The U. S. Environmental Protection Agency and the Corps of Engineers represent the Federal role in the study. The Georgia Environmental Protection Division provides the State involvement and the Atlanta Regional Commission is the regional participant and represents 46 local governments in the Atlanta area. The wastewater and water supply plans[1,2] outline the various expansions and modifications required to meet the area's needs for water supply and sewage treatment from the mid 1970's to the year 2000.

Atlanta, Georgia is a major city in the southeastern United States and serves as the transportation center of the southeast. Many large corporations maintain regional offices in Atlanta. Also, most Federal agencies, including the Environmental Protection Agency, have regional offices in Atlanta. Industrial operations in the area are generally low water users;

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consequently, the approximate 7.80 m^3/s (178 mgd) of wastewater from the region's 1.7 million population is primarily domestic sewage.

THE ATLANTA REGION

The Atlanta region is located about 335 meters (1100 feet) above sea level. Although it is not as significant or spectacular, one of the major drainage divides of the United States extends from a northeasterly direction through the City to a southwesterly direction. East of this divide water eventually flows into the Atlantic Ocean; whereas, the water west of this divide flows into the States of Florida and Alabama and into the Gulf of Mexico. The upper portion of the South River is formed in Atlanta on the east side and the Flint River begins on the southern edge of the City. The Chattahoochee River flows to the west of the City. On all three of the rivers downstream of the Atlanta metropolitan area are significant water users which include recreational lakes and community water supply system intakes.

The Atlanta metropolitan area with a population of about 1.7 million is one of the larger population centers in the United States which is located inland from the sea and on a relatively small river. The Chattahoochee River flow is regulated by the U. S. Army Corps of Engineers' Buford Dam - Lake Lanier hydroelectric operation located some 77.2 km (48 miles) upstream of Atlanta. From that 77.2 km (48 miles) reach of the Chattahoochee River four local governments, including Atlanta, take water to supply the water systems which collectively serve one-third of Georgia's population. This regulated stream, with fluctuating flows caused by the upstream hydroelectric operation compounded by significant withdrawals by water supply intakes, had flows in the early 1970's that dropped as low as 17.0 m^3/s (600 cfs) further compounding severe water quality problems downstream of Atlanta. The wastewater management plan and the water supply plan have been developed to resolve many of the existing problems and to provide the required systems to meet the needs of the future.

The Atlanta region encompasses seven counties. These seven counties extend over 5330 square kilometers (2058 square miles). Although the 1977 population is approximately 1.7 million, it is predicted that by the year 2000 approximately 3.5 million people will live in the Atlanta region.

PRESENT STATUS OF WASTEWATER TREATMENT FACILITIES

All wastewater treatment systems in the Atlanta Metropolitan Area are modern facilities. In the seven-county metropolitan area, a total of 24 major treatment facilities with design capacities ranging from 0.044 m^3/s to 5.26 m^3/s (1.0 mgd to 120 mgd) are currently in operation. Their total capacity is 12.5 m^3/s (286 mgd) and collectively they are treating a flow of 7.80 m^3/s (178 mgd).

Although these systems were designed to achieve secondary biological treatment, many of them were designed and constructed prior to the establishment of regulations by the U. S. Environmental Protection Agency in 1973 which defined uniform national standards for secondary treatment. This definition requires that all municipal wastewater treatment facilities achieve at least 85 percent removal of influent BOD_5 and suspended solids, and produce effluent quality with BOD_5 and suspended solids concentrations not exceeding 30 mg/l each, with effluent pH in the range of 6-9 standard units, and fecal coliform not exceeding a geometric mean of 200/100 ml. All of the treatment facilities in the Metropolitan Area provide biological treatment, but some of these facilities do not achieve the secondary treatment limitations.

Conditions which prevent the consistent achievement of secondary treatment standards are inadequate operator training, staffing, and maintenance of facilities; solids handling; presence of industrial wastewaters; and infiltration, inflow, and combined sewers.

Most local government officials in Georgia understand the importance of having sewer systems and wastewater treatment facilities in their communities, but many do not understand the importance of operation and maintenance for those facilities. Thus the budgets for plant maintenance and salaries for operators are often low. Hiring and retaining qualified operators, then training and motivating them to get optimum efficiency from the treatment works they operate, are significant problems which are being resolved very slowly.

Solids handling is a problem which plagues a few Metropolitan Atlanta treatment facilities. Two facilities have anaerobic digesters which ceased to function in recent years due to long-term accumulations of grease, grit and heavy metals from industries, with the only solution to empty the digesters and place their contents in lagoons or landfills. In early 1977 the first and third largest treatment facilities, which incinerate a mixture of undigested and digested dewatered sludge, had their natural gas fuel supplies interrupted due to a national gas shortage in critically cold weather. The partially treated sludge had to be landfilled until natural gas service was restored. The largest metropolitan facility operated poorly during most of 1976 since several of its primary settling tanks were out of service during construction, causing significant solids losses.

Industrial wastes which enter the sewer systems without adequate pretreatment cause upsets of biological treatment processes. All local governments have undertaken programs to identify industrial contributors and determine their wastewater characteristics. These programs will eventually lead to the necessary pretreatment or the elimination of incompatible wastes.

Sewer systems in the suburban areas surrounding the City of Atlanta are relatively new and generally do not have severe problems with infiltration and inflow. However, most engineering studies completed recently have shown that problems with infiltration/inflow exist in some portions of every sewage treatment plant service area. Certain portions of the City of Atlanta are served by combined storm and sanitary sewers, some of which were built as many as seventy years ago. There are four combined sewer overflow points in the South River Basin[3]. Federal construction grants are being offered to assist local governments in rehabilitating sewers which have excessive infiltration and inflow. In the South River Basin, grants will be made available to provide secondary treatment for combined sewer overflows resulting from rainfalls of up to 1.8 cm (0.7 in).

Table 1 is a compilation of relevant information on existing treatment works with capacities exceeding 0.22 m^3/s (5 mgd), and Table 2 lists some average influent and effluent wastewater characteristics for these facilities for 1976.

WATER QUALITY MANAGEMENT PLANS

Section 303(e) of the Federal Water Pollution Control Act of 1972 requires all states to develop comprehensive water quality management plans for each major river basin. These documents must contain inventories of all point wastewater sources, assessments of the effects of those pollution sources on the quality of receiving waters, and waste load allocations for each pollution source. This information is then used to develop effluent limitations for discharges and to establish priorities for pollution abatement efforts.

Prior to the passage of Public Law 92-500, the State of Georgia had already begun the function of water quality management planning. Programs undertaken by the State began with documentation of serious water pollution problems in the three stream systems receiving the most wastewater in the Atlanta Metropolitan Area: the Chattahoochee, Flint and South Rivers. Following this documentation, the State undertook programs to measure certain physical, chemical, and biological characteristics of the streams so that mathematical models for certain water quality parameters could be developed. Dissolved oxygen depletion was considered to be the most serious water quality problem in all three of these streams. All three streams had organic and nitrogenous waste loads sufficiently high to cause septic conditions during periods of warm weather and low stream flows, even with good stream reaeration characteristics.

TABLE 1. SYSTEM DESCRIPTION - MAJOR TREATMENT WORKS
ATLANTA, GEORGIA METROPOLITAN AREA

Facility	Type of Treatment	Treatment Capacity		Type of Sewer System	Receiving River
		mgd	m^3/s		
Flint River WPCP #1	AS	6	0.26	SEP	Flint
Flint River WPCP #2	AS & TF	9	0.39	SEP	Flint
South River WPCP	AS & TF	18	0.79	COM	South
Intrenchment Creek WPCP	TF	20	0.88	COM	South
Snapfinger WPCP	AS & TF	22	0.96	SEP	South
Big Creek WPCP	AS	6	0.26	SEP	Chattahoochee
Chattahoochee WPCP	AS	10	0.44	SEP	Chattahoochee
Clayton WPCP	AS	120	5.26	COM	Chattahoochee
Utoy Creek WPCP	AS	30	1.31	COM	Chattahoochee
South Cobb WPCP	AS	8	0.35	SEP	Chattahoochee
Camp Creek WPCP	AS	15	0.66	SEP	Chattahoochee

ABBREVIATIONS:
WPCP - Water Pollution Control Plant
AS - Activated Sludge
TF - Trickling Filter
SEP - Separated Sanitary Sewers
COM - Combined Storm & Sanitary Sewers

TABLE 2. 1976 AVERAGE PERFORMANCE - MAJOR WASTEWATER TREATMENT WORKS ATLANTA, GEORGIA METROPOLITAN AREA

WPCP Facility Name	Treated Flow mgd	Treated Flow m^3/s	Inf. BOD_5 mg/l	Eff. BOD_5 mg/l	Inf. SS mg/l	Eff. SS mg/l	Eff. pH SU	Effluent Fecal Coliform No./100 ml
Flint River #1	2.5	0.11	194	34	140	34	7.2	420
Flint River #2	8.3	0.36	202	43	330	94	7.6	75
South River	14.3	0.63	210	27	149	47	7.3	230,000
Intrenchment	12.4	0.54	202	35	122	30	6.9	1,400,000
Snapfinger	15.0	0.66	184	26	194	20	6.8	60
Big Creek	2.8	0.12	120	11	124	9	6.7	4
Chattahoochee	9.9	0.43	238	63	460	114	6.8	76
Clayton	79.7	3.49	146	41	171	74	6.6	790,000
Utoy Creek	13.7	0.60	150	23	177	42	7.0	540
South Cobb	8.3	0.36	153	40	143	48	6.4	140
Camp Creek	4.4	0.19	95	22	119	28	6.5	---

ABBREVIATIONS:
Inf. - Influent
Eff. - Effluent
SS - Suspended Solids
SU - Standard Units

The radiotracer technique developed by Tsivoglou[4], which is the only direct measurement of reaeration in natural streams, was used in the model development on all three of these rivers. The technique[5] uses three tracers simultaneously: krypton-85, as a dissolved gas, is the tracer for dissolved oxygen; tritium, as water molecules, is the tracer for dispersion; and rhodamine - WT fluorescent dye provides information on time of flow. The ratio of relative gas transfer capabilities of krypton and oxygen, measured as their respective reaeration coefficients, is 0.83, with krypton transfer being the slower process. By measuring of the loss of the krypton tracer to the atmosphere and accounting for in-stream dispersion, a direct correlation to oxygen transfer is obtained.

The mathematical model used for each stream was a modified Streeter-Phelps equation which accounted for reaeration, carbonaceous oxygen demand, nitrogenous oxygen demand, and benthic oxygen demand. The model was steady-state and one-dimensional. The model for each stream was calibrated to verify measured field data and was then used to predict the responses of the stream under future conditions of wastewater flows and treatment levels. Computer programs were used to test a wide variety of alternatives in each of the models.

Uniform effluent limits were established for all major wastewater discharges in the Flint River and the South River Basins. The effluent limitations were: BOD_5 not to exceed 10 mg/l, ammonia nitrogen not to exceed 2 mg/l, total phosphorus not to exceed 1 mg/l, and dissolved oxygen not less than 6 mg/l. The phosphorus limitation was established for treatment works in the South River Basin to reduce eutrophication of a popular recreational lake some 64 km (40 miles) downstream. The phosphorus limitation was imposed on treatment works in the Flint River Basin so that reasonably high treatment would be applied to upgrade the quality of the Flint River which is used for water supply downstream from the Atlanta Metropolitan Area.

Both the Flint River and the South River are unregulated streams and are quite small, with their headwaters rising in the Metropolitan Area. The Chattahoochee River, however, with a drainage area of 3760 sq. km. (1450 sq. mi.) at Atlanta is a highly regulated stream due to the generation of hydroelectric power upstream from the Metropolitan Area. The Chattahoochee River receives wastewater discharges from the sewer systems of three county governments and the City of Atlanta. In addition to its widely fluctuating flow regime, the Chattahoochee River has significant withdrawals for local water supplies and is used as once-through cooling water for two major steam electric generating plants which serve the area. Taking into account the effects of all of these conditions, the State again constructed mathematical water quality models of the Chattahoochee River and determined that a minimum of 21.2 m^3/s (750 cfs) of water would be needed in the Chattahoochee River just downstream from the City of Atlanta's water supply pumping station in order to maintain an acceptable stream quality for a "fishing" classification. In order to meet fishing standards at a minimum flow of 21.2 m^3/s (750 cfs) on the Chattahoochee River, it was necessary to establish a uniform treatment level through the mathematical modeling process for all major dischargers, assuming a minimum river flow of 21.2 m^3/s (750 cfs). All dischargers were required to achieve a level of treatment higher than the EPA established secondary treatment standards. The effluent quality to be met by the dischargers on the Chattahoochee River is: BOD_5 not to exceed 17 mg/l, ammonia nitrogen not to exceed 5 mg/l, and dissolved oxygen not less than 6 mg/l.

The results of the mathematical modeling and waste load allocation processes for these three rivers were incorporated into the State's water quality management plans required under Section 303(e) of PL 92-500. These limitations were also written into the discharge permits for all affected treatment facilities and information from these studies was incorporated into the local government discharge ranking system which is used to prioritize projects for Federal construction grants. Through the planning process, and as a result of the important water uses downstream from the Atlanta area in all directions, it has been determined that every publicly owned treatment facility, whether presently existing or planned for the future, will have to achieve effluent quality higher than secondary treatment.

DETAILS OF FUTURE SYSTEMS

Solutions to meeting the necessary water quality standards and complying with the effluent limitations established by the State have been developed by several of the major dischargers through the facilities planning process of the Federal construction grant program. Facilities planning, as required by Section 201 of PL 92-500, has not been completed for all sewer service areas within the Atlanta Metropolitan Area, but a considerable portion of such planning has been done. The trend in the metropolitan area is toward construction of large regional treatment works with extensive service areas, since these systems offer significant economies of scale. A variety of technical solutions to meet the required effluent limitations has been developed by consulting engineers retained by the local governments.

Two treatment facilities discharging to the Chattahoochee River will be the first to be upgraded and expanded in the Atlanta area. The Chattahoochee Water Pollution Control Plant (WPCP), which receives wastewater from two cities, and unincorporated areas of two counties, has an expansion from 0.44 m^3/s (10 mgd) to 0.88 m^3/s (20 mgd) under construction. The facility, which currently utilizes a conventional activated sludge process and anaerobic sludge digestion, will convert its aerobic digesters to additional activated sludge basins. Plastic media nitrification towers with pH adjustment capability will be added to reduce ammonia concentrations following the activated sludge process. Expected final effluent quality is: BOD_5 not exceeding 17 mg/l, $NH_3(N)$ not exceeding 5 mg/l, and suspended solids not exceeding 30 mg/l. Sludge will be anaerobically digested and dewatered in filter presses, with the filter cake being incinerated. The incinerator will be fueled by digester gases.

The South Cobb WPCP, which treats wastewater from five cities and portions of two counties, will be expanded from 0.35 m^3/s (8 mgd) to 1.05 m^3/s (24 mgd). The existing conventional activated sludge system will be expanded and converted to use pure oxygen aeration, combining carbonaceous removal and nitrification in one process. Pure oxygen gas will be generated by a 2900 kg/day (32 ton/day) cryogenic plant, and liquid oxygen storage will provide a backup supply. The South Cobb WPCP will meet the same final effluent limits as the Chattahoochee WPCP. Undigested sludge will be gravity thickened, heat dried in rotating kilns, pelletized and bagged for use as a fertilizer and soil conditioner.

All wastewater currently being discharged to the Flint River will be removed. Effluents from the Flint River WPCP #2 and a smaller facility nearby will be disposed on land by spray irrigation. The effluents from the Flint River WPCP #1 and another smaller facility will be pumped with effluent from two facilities in the South River Basin to the Chattahoochee River. These actions will be particularly beneficial since the Flint River Basin is the most severely water quality limited and the smallest of all of the major drainage basins in the Metropolitan Area.

Engineering analysis showed that it would be more cost-effective (cost-effectiveness is determined by comparing the present worth of treatment alternatives) to construct a land application system than to provide the advanced levels of treatment required for stream discharge. The Flint River WPCP #2 will be expanded to provide secondary treatment for future population needs and the effluent will be sprayed onto an area of land which is tributary to the surface water supply of the same authority which owns this plant. Thus, this local government, which is predicted to have future water shortages, will actually be recycling a large portion of its wastewater, since approximately 90 percent of the water which falls onto the spray fields will ultimately find its way back into surface streams through groundwater channels. The authority will also grow trees in the land application area which will be harvested and sold to pulp and paper companies. The project will consist of pumping an average of 0.85 m^3/s (19.5 mgd) of wastewater which has received conventional secondary treatment to a storage reservoir which will have sufficient volume to store 12 days of waste flow. The water will then be pumped onto wooded areas of sandy clay soil through a sprinkler distribution system approximately 488,000 m (1,600,000 ft.) in length. The year-round application rate will average 6.35 cm (2.5 in.) per week and the total wetted area of land will be approximately 955 ha (2360 acres). The authority will have to acquire approximately 1420 ha(3500 acres) of land to allow adequate buffer areas around existing homes and roads.

In the South River Basin, the largest existing facility is the Snapfinger WPCP which will be upgraded and expanded to 1.58 m^3/s (36 mgd) to meet future population needs, and will continue to discharge to the South River. To meet the required effluent limits of BOD_5 = 10 mg/l, $NH_3(N)$ = 2 mg/l and total phosphorus = 1 mg/l, the facility will employ the following processes: chemical precipitation in primary clarifiers using lime for removal of organic materials; aeration basins for biological nitrification to remove ammonia; post-chemical precipitation for phosphorus removal; and dual-media filtration for suspended solids removal. Post-chemical sludge will be classified by centrifuges, recalcined, and reused. Primary and nitrification sludges will be filter pressed, with the filter cake being incinerated.

The City of Atlanta, largest local government in the metropolitan area, has wastewater treatment facilities in the Flint River, South River, and the Chattahoochee River Basins. Faced with providing advanced wastewater treatment (effluent limits identical to those from the Snapfinger WPCP) for its South River, Flint River #1, and Intrenchment Creek WPCP's, Atlanta selected a plan which will divert the effluents of those facilities a distance of some 16.1 km (10 mi.) through underground tunnels, back to the Chattahoochee River from which the water originated. In doing this, Atlanta has the advantage of providing a lower level of treatment (BOD_5 = 15 mg/l, $NH_3(N)$ = 4 mg/l, and no limit on phosphorus) for these effluents than it would have if it had left them in the Flint and South Rivers. The City will have substantial economic benefits by reducing the annual operating costs below those which would be required for operating more advanced treatment systems. In addition, the Three Rivers proposal will provide retention and partial treatment for wastewater from three combined sewer overflows in the South River Basin.

All owners of wastewater treatment and collection systems in the Atlanta Metropolitan Area, other than those discussed in the preceding paragraphs, are in various stages of detailed planning to determine what actions may be necessary to meet future needs and to comply with effluent limits established through the State's planning processes. Availability of grants from the U. S. Environmental Protection Agency to pay for 75 percent of the costs of planning, design, and construction assures the continued interest and cooperation of the local governments in these programs.

REGIONAL-AREAWIDE WASTEWATER MANAGEMENT PLAN

The Atlanta Regional Commission(ARC) is the regional planning and intergovernmental coordinating agency for 46 local governments in the Atlanta Metropolitan Area. The Commission has a full-time technical staff which has the responsibility for preparing regional plans for transportation, land use, health, social services, water supply, and wastewater management. However, ARC does not have the authority or responsibility to implement the plans. The 46 local governments must implement various plans developed by ARC; however, the intergovernmental cooperation that is essential to the implementation of a plan is often accomplished through ARC.

Recognizing the complexities of wastewater management and the fact that natural drainage areas often cross several local government boundaries, the ARC requested assistance from the Federal and

State governments in the development of a comprehensive plan. In 1973 the joint effort was initiated. While the ARC's technical staff emphasized coordination of local governments within the region and the solution of their problems, the Federal and State agencies placed emphasis on evaluating the impact of ARC's wastewater and water supply plans on areas outside the region.

The Federal agencies (EPA and Corps of Engineers) conducted portions of the study that produced statistical data to guide the selection of the most cost-effective alternatives. An in-depth evaluation of the stream flows of the area was made and predictions for minimum flows were developed. The State conducted stream studies to verify existing water quality and through computer modeling determined allowable waste loads. Utilizing the construction grant funds provided to the State under Section 201 of PL 92-500, the State has worked with the local governments to initiate implementation of the recommended modifications and improvements to the publicly owned treatment works in the region.

During 1976 and in accordance with Section 208 of PL 92-500, the State designated the region served by ARC for the development of an areawide wastewater management plan and designated ARC as the agency to develop the plan. Many of the improvements needed in the region's system of point wastewater sources have begun to be implemented as a result of State requirements and the regional study conducted between 1973 and 1976. Efforts under the 208 study will concentrate on development of a plan to control or minimize non-point source pollution. Pollution from urban runoff, which includes soil-erosion, sewage from combined sewers, storm drainage from commercial districts, infiltration and inflow to sanitary sewers, and debris from the streets, will be studied. Alternatives will then be recommended to reduce non-point source pollutants. Preliminary studies indicate the non-point source pollutants contribute as much as 50 percent of the pollutional load to the streams in Atlanta region during periods of high rainfall. Since the existing point sources receive biological treatment already, the additional reduction from point sources will be small and very expensive which makes it imperative that the 208 study produce cost-effective alternatives to reduce and control non-point source pollution.

WATER SUPPLY PLAN

An adequate and dependable water supply for any community is essential for the health, safety, and economic well-being of the region. In the Atlanta region the geological formations are such that groundwater is unavailable. Therefore, the region is dependent on surface waters as a source for water supply. The drainage divide through the Atlanta region, with the Chattahoochee River being the only river which flows through the region, further limits available water. The necessity of maintaining a minimum streamflow past Atlanta for the assimilation of treated wastewater from the region further reduces the available supply.

With the projected population of 3.5 million by the year 2000 and the established minimum flow for assimilation of treated wastewater of 21.2 m^3/s (750 cfs), the development of a plan to provide for realistic alternatives to meet the water needs of the region had become critical by the mid 1970's. Since the Chattahoochee River has a hydroelectric facility about 77.2 km (48 miles) above Atlanta, an evaluation was made of the reservoir to determine the maximum sustained yield that could be provided during drought conditions. The maximum sustained yield for a drought condition comparable to the most severe drought of record was determined to be 51.0 m^3/s (1800 cfs). In order for the region's peak day demand by the year 2000 of 29.7 m^3/s (1050 cfs) to be met under drought conditions, two alternatives have been identified as realistic. One alternative is to construct a smaller re-regulation dam about 9.65 km (6.0 mi.) downstream of the existing hydroelectric facility. A second alternative is to modify the operation of the existing hydroelectric facility to provide a constant flow of at least 51.0 m^3/s

(1800 cfs) at all times. Both of these alternatives are presently under further economic and environmental study. In the meantime, a slight modification in the operation of the hydroelectric facility has assured the region of the needed water until the 1985-1987 period.

FEDERAL-STATE-REGIONAL EFFORTS

The comprehensive three year effort by Federal, State, and Regional agencies has produced implementable plans for management of wastewater and water supply systems for the Atlanta region through the year 2000. Local governments in the region must now implement the plans under the guidance of the State and with the assistance of ARC. Local governments must begin requiring conservation of water through education of their citizens and establishment of new plumbing codes. The Federal agencies must continue to participate in these efforts to assure an adequate water supply to the region and an approvable non-point source pollution control strategy. In the final analysis, it has been determined that the water resources of the Atlanta region may be the controlling or limiting factor for the region's future instead of energy or other resources.

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REGIONAL SOLUTIONS TO DOMESTIC WASTEWATER MANAGEMENT

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ABSTRACT

Regional solutions to wastewater collection and treatment are discussed in terms of encouraging their adoption. Problems encountered in implementing regionalization are identified as are the many advantages that can be gained if these problems are overcome. A number of important issues that should be considered in any regionalization effort are discussed.

INTRODUCTION

The creation of regional agencies to collect and treat wastewater is usually a difficult and painful process. There are many reasons for this-but-the following are among those most frequently encountered. Existing wastewater collection and treatment jurisdictions are very reluctant to give up their responsibilities-and-if they do, often maintain bitterness over lost responsibilities for a long period of time. And, a new regional agency inherits all of the politically based hostilities of its constituencies. If these and other problems can be overcome, there can be real advantages to regionalizing wastewater management systems.

BACKGROUND

Much of the time regional solutions to regional problems are defended on the basis of their value to protecting the environment. This is because a large number of environmental concerns do not respect the artificial boundaries of cities and counties as such; they must be attacked regionally on the basis of river basins or air sheds for example. However, the most compelling arguments on behalf of regional wastewater management may be economic and political rather than environmental.

Regionalization in the wastewater field is not new. There are many agencies in large urban areas of the U.S.A. as well as Europe that have long histories in water pollution control. Recent Federal legislation has encouraged regionalism by promoting the development of regional waste management plans and by encouraging the most cost-effective solution to wastewater collection and treatment.

Concerning the latter, it just didn't seem economical to have two wastewater treatment plants, perhaps even across the street from one another, separated by jurisdictional boundaries. Historically, most large urban areas of the U.S.A. had many--many highly independent political jurisdictions. Over the years some progress was made in waste management through the formation of semi-autonomous sewer districts throughout an area. Further, intercommunity sewerage agreements were started between a number of area communities in particular urban centers. But, while these steps represented some improvement, many problems remained.

REGIONALIZATION CONSIDERATIONS

A successful regional wastewater management agency usually has the following characteristics:

1. The agency fills a clearly defined and recognized need.

2. There is a strong personal commitment to regionalization on the part of the elected officials and key members of institutions that are involved.

3. A sound financing program exists which is equitable for all users.

There are two major concepts of regionalization. The first concept is one where the regional agency acts as a wholesaler of services to existing jurisdictions. These jurisdictions then continue to deal with the public on a "retail" level. The second concept is a complete consolidation, wherein the regional agency provides the service and deals with the public as a customer. In creating a wholesale/retail concept of regionalization, one of the more difficult aspects is to determine which resources belong to the regional agency and which to the surviving local jurisdictions.

A most important consideration is that of properly structuring the governing Board or Commission to manage the agency. The governing body should be responsive to the public if the agency has taxing authority then it may need equal representation, i.e. "one man, one vote". If the smallest jurisdiction in a regional authority has one vote, then the larger jurisdictions must have a proportionately larger vote.

Financial considerations leading to regionalization decisions include a look at the status of each agency in terms of whether they will benefit from regional service and a look at the equity each may have in the present sewers and treatment plants. Future funding for regional agencies eases financial problems because the Federal and State governments providing grants can deal with one entity. It avoids the technical and political problems of trying to resolve conflicts between different agencies not wanting to become part of a regional system.

But, at EPA, if we think a regional system is the cost-effective way of providing wastewater collection and treatment of domestice waste we use economic incentives to force the issue. That is, we refuse to award the 75% construction grant if the political jurisdictions don't get together. Or, if a city was awarded a grant on the basis it would serve as a regional facility--and then--it refuses to accept another cities waste for political reasons (such as insisting on annexation first), we refuse to make payments on the grant or to award future grants. These financial incentives plus an incentive often used by State governements--a ban on new connections to existing sewers--usually forces a favorable regionalization decision.

Another problem to consider in regionalization decisions is one of people. Preserving the identity of workers, managers and public board members who represent agencies that are affected by regional plans is important. It is often good practice to accept existing staff and retrain them if necessary.

ADVANTAGES

There are a number of advantages possible with regional wastewater management systems. First, there are economies of scale, both for the capital investment and for the operation and maintenance expenses. Also, cities may be able to transfer their debt for capital costs to a regional authority where it can be handled easier.

There are water quality advantages to regional system. It can eliminate multiple discharges that are at the wrong point and are causing water pollution. Regionalization can provide more flexibility in treatment plant operation and--being a larger system--probably can support a budget for better operation and maintenance. Consolidation of technical and financial resources could allow for problem solving; i.e. some R&D work. Finally, there is a likelihood that the growth and development of an urban area will be more orderly than if small individual jurisdictions were providing sewerage service.

In summary, forming regional wastewater management systems can be cost--effective, provide for compliance with water quality standards, and it can provide a high level of political and managerial experience and accountability.

URBAN RUNOFF POLLUTION CONTROL TECHNOLOGY OVERVIEW

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ABSTRACT

This Overview describes the major elements of the Urban Runoff Pollution Control Program. Problem Definition, User Assistance Tools, Management Alternatives and Technology Transfer are covered, including some of the highlights of the Program's future direction and products from over 150 of its research projects. References are cited for completed Program reports, ongoing Program projects, and in-house documents. Capital cost comparisons for storm and combined sewer control/treatment are given, along with a specific example of cost-effective solution for urban runoff pollution control by in-line storage in Seattle. In a study done in Des Moines, using a simplified receiving water model, four control alternatives were compared, considering cost and effectiveness in terms of a frequency of D.O. standard violations.

INTRODUCTION

Control and treatment of stormwater discharges and combined sewage overflows from urban areas are problems of increasing importance in the field of water quality management. Over the past decade much research effort has been expended and a large amount of data has been generated, primarily through the actions and support of the U.S. Environmental Protection Agency's Storm and Combined Sewer Research and Development Program.

The products of the Program (Figure 1) are divided into the following areas, common to the major elements of Combined Sewer Overflow Pollution Control, and Sewered and Unsewered Runoff Pollution Control: Problem Definition, User Assistance Tools (Instrumentation, Computers), Land Management, Collection System Control, Storage, Treatment, Sludge and Solids, Integrated Systems, and Technical Assistance and Technology Transfer.

Table 1 breaks down these categories into more specific elements There have been about 150 projects under the Program. References are cited for completed Program reports (numerically indicated), ongoing Program projects (indicated by "P" numbers), and in-house and miscellaneous documents (indicated by "R" numbers).

PROBLEM DEFINITION

The program starts with "Problem Definition" broken into "Characterization" and "Solution Methodology" (Figure 2).

The background of sewer construction led to the present urban runoff problem. Early drainage plans made no provisions for storm flow pollutional impacts. Untreated overflows occur from storm events giving rise to the storm flow pollution problem.

Simply stated the problem is:

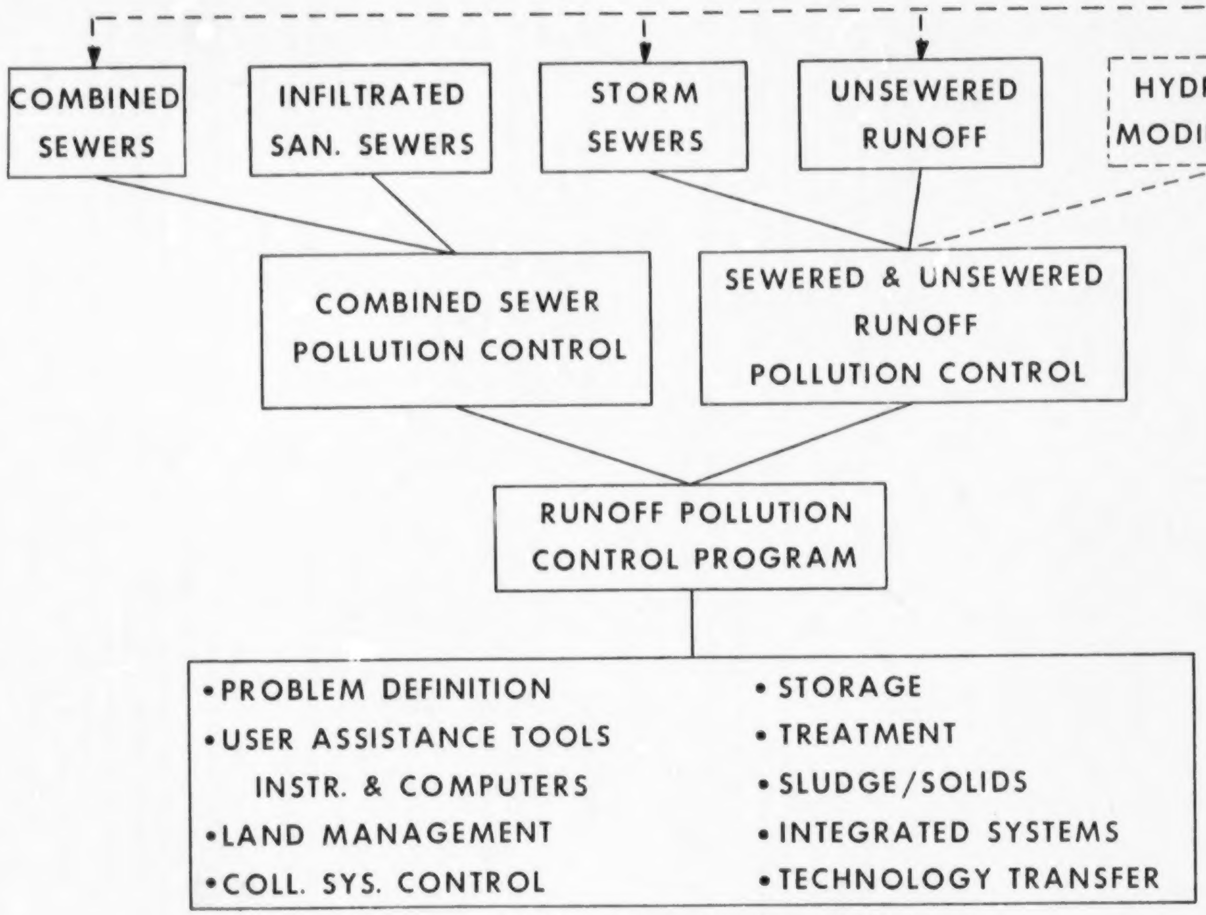

Figure 1. EPA Storm and Combined Sewer R&D Program

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CATEGORIES	INITIATED/ACCOMPLISHED	ONGOING
PROBLEM DEFINITION		
Characterization	Prelim. appraisals CSO/SW prob., CSO/SW char., deicing, sed./eros., loading factors, rec. water impacts, nat'l assess. impacts/costs	-Direct rec. water/source loading factor analysis -Dev. S&CS strategy document -Analyze optimum S&CS/DWF T/C combinations
Solution Methodology	SOTA S&CS tech., plan/select guide, conduct of SW studies, SOTA's sed./eros. & deicing control, unit cost factor dev.	
USER ASSISTANCE TOOLS		
Instrumentation	Raingage, flow measuring, sampling, monitoring, control	-Verif. magnetic flowmeter for simultaneous press/gravity flow meas. (supplement) -Demo in-situ TOC anal. & storm flow sampler (Syracuse-supplement) -Dev. syst. analysis program for quantification/handling of CSO sludge/solids -Dev. autom. oper. model for real-time control w/rain fall predict. -Hydr. model for steep slopes (supplement)
Simulation Models	Simplified, detailed/complex, operational, dissemination	
LAND MANAGEMENT	NON-STRUCTURAL:	
Retentien/detention	Land use planning, upstream impoundment (structural), porous pavement	-Eval. effect of sed. trapping efficiency of off-stream det./ret. -Demo in-situ hydrophobic substance -Demo sed./eros. control techniques in SE USA (supplement) -Demo improved street cleaning practices
Enforced controls	Air pollution, eros./sed., cropping, berms, chemical use, hydrophobic, deicing	
Neighborhood sanitation	Street cleaning, solid waste management	
COLLECTION SYSTEM CONTROLS		
Sewer separation	May require separate treatment system	-Demo periodic sewer flushing: CSO 1st flush relief -Eval. strength increases from sulfur impregnation of concrete pipe -Dev. autom. operational model for real-time control w/rainfall predict.
Runoff inlet/catch basin	Effectiveness eval., improved design, cleaning	
Sewers	"First flush" relief: flushing/cleaning, new designs (low flow carrying vel. and added storage), I/I prevent and control (with manuals), polymers to increase capacity	
Improved regulators and tide gates	Positive control, increased diversion to treatment, swirl & helical, fluidic reg.	
Remote monitoring with supervisory control	Provides storage/discharge options	
Fully automated control	Automates storage/discharge options	
STORAGE		
In-Line	Use excess sewer capacity	
Off-Line	Surface (rect. tanks, earthen basins, abond. quarries), subterrain. (natural formations, silos & tunnels), underwater, solids impacts	
TREATMENT		
Physical treatment with and without chemicals	Fine screening, microstraining, sed., filt., dissolved air flotation, swirl/spiral separation	-Demo fine screening/microstraining, swirl reg. and high-rate disinf. by ClO_2/Cl_2 and mixing incl: resid. toxic/carcinogenic Cl_2 comp. viral disinf. eval.) (Syracuse-supplement) -Feas. of land disposal (Envirex-supplement)
Biological treatment	Contact stabilization, trickling filters, lagoons, rotating biological contactors. Generally assoc. w/continuously operated plant	
Physical-chemical	Precipitation, filtration, adsorption, ion exchange, break-pt Cl_2	
Disinfection	Chlorination, hypochlorination, ozonation, chlorine dioxide, on-site gen., high-rate, mixing, microorganism indicator study (pathogen, virus), 2-stage	
Land disposal	Marsh land	
SLUDGE/SOLIDS		
Characterization/Quantification	Classification requirement, treatability, vital parameters, amount, solids content	-Evaluate: methods of ultimate disposal of WWF solids and impacts of WWF sludges/solids on DWF plant (Envirex-supplement)
Treatment handling schemes	On-site vs DWF trmt., land disposal	
Process evaluations	Thickening, digestion, centrifugation, vacuum filtration, incineration	
INTEGRATED SYSTEMS (Varies)	Most common master plan approach	
Storage/treatment	Pump-back, sed. in storage, disinfection, break-even econ. w/treatment	
Dual use WWF/DWF (storage/treatment	Lagoon storage/treat., HRTF, contact stab., P-C, hi-rate filter, equalization, combined sewers	
Control/treatment/reuse	Treatment-park, new town devel. (land mgmt-treat), storage/treat. marina feas., lakelets for storage/treat./reuse	
TECHNICAL ASSISTANCE AND TECHNOLOGY TRANSFER		-Continuous (15% - 20% of prog. time) NOTE: Anticipate increase in EPA construction grant (201) and planning grant (208) assist. due to CSO emphasis
Consultation to Fed., state, local govts. and quasi-govt. agencies	EPA, OAWP (needs surveys): EPA TT (seminars, film production); EPA Reg. receiving H_2O impact studies; EPA Hq and Regions on 201/208 studies and seminars; Reg. V on 108 grants; NSF, DOT, OWRT (reviews, confer, steering committees); CEQ & NCWQ (proposal and rept reviews, joint nat'l assess. projects)	
Public Inf. Requests	Documents, lectures, films, confer., articles, reports, example methodology for prob. solution, conf. moderator, prog. committees	
Consultation to foreign govt. and international confer.	TT (France, Japan, Denmark, Norway, Sweden, Canada, Netherlands, Australia, New Zealand); Canadian (TAG) IJC (steering com.) IAWPR (confer. & prog. committees); various conferences and publications	
In-House seminars	Various tech. areas; overviews	
SWMM	Short course,; user's assistance manuals/dissemination	
Higher Education	SCS prog. university course man.	
Planning/design/SOTA/assess manuals and extramural publications	Overall prog. concepts, sol. method, sampling/anal., costs, specific processes	

Table 1. Summary Storm and Combined Sewer Program

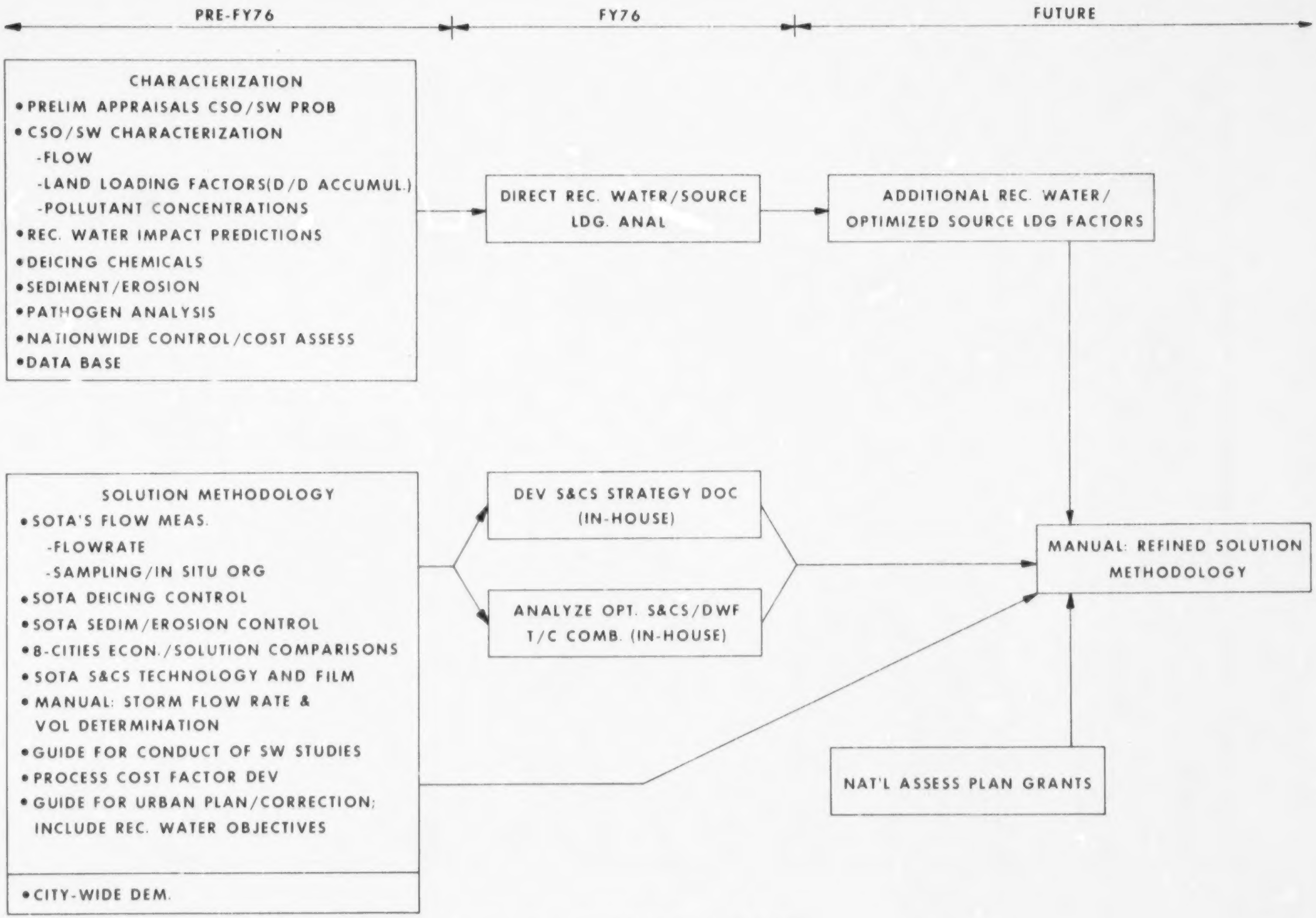
PRE-FY76
FY76
FUTURE
CHARACTERIZATION
•PRELIM APPRAISALS CSO/SW PROB
•CSO/SW CHARACTERIZATION
-FLOW
-LAND LOADING FACTORS(D/D ACCUMUL.)
-POLLUTANT CONCENTRATIONS
•REC. WATER IMPACT PREDICTIONS
•DEICING CHEMICALS
•SEDIMENT/EROSION
•PATHOGEN ANALYSIS
•NATIONWIDE CONTROL/COST ASSESS
•DATA BASE
DIRECT REC. WATER/SOURCE LDG. ANAL
ADDITIONAL REC. WATER/ OPTIMIZED SOURCE LDG FACTORS
SOLUTION METHODOLOGY
•SOTA'S FLOW MEAS.
-FLOWRATE
-SAMPLING/IN SITU ORG
•SOTA DEICING CONTROL
•SOTA SEDIM/EROSION CONTROL
•8-CITIES ECON./SOLUTION COMPARISONS
•SOTA S&CS TECHNOLOGY AND FILM
•MANUAL: STORM FLOW RATE & VOL DETERMINATION
•GUIDE FOR CONDUCT OF SW STUDIES
•PROCESS COST FACTOR DEV
•GUIDE FOR URBAN PLAN/CORRECTION; INCLUDE REC. WATER OBJECTIVES
•CITY-WIDE DEM.
DEV S&CS STRATEGY DOC (IN-HOUSE)
ANALYZE OPT. S&CS/DWF T/C COMB. (IN-HOUSE)
MANUAL: REFINED SOLUTION METHODOLOGY
NAT'L ASSESS PLAN GRANTS

Figure 2. Problem Definition

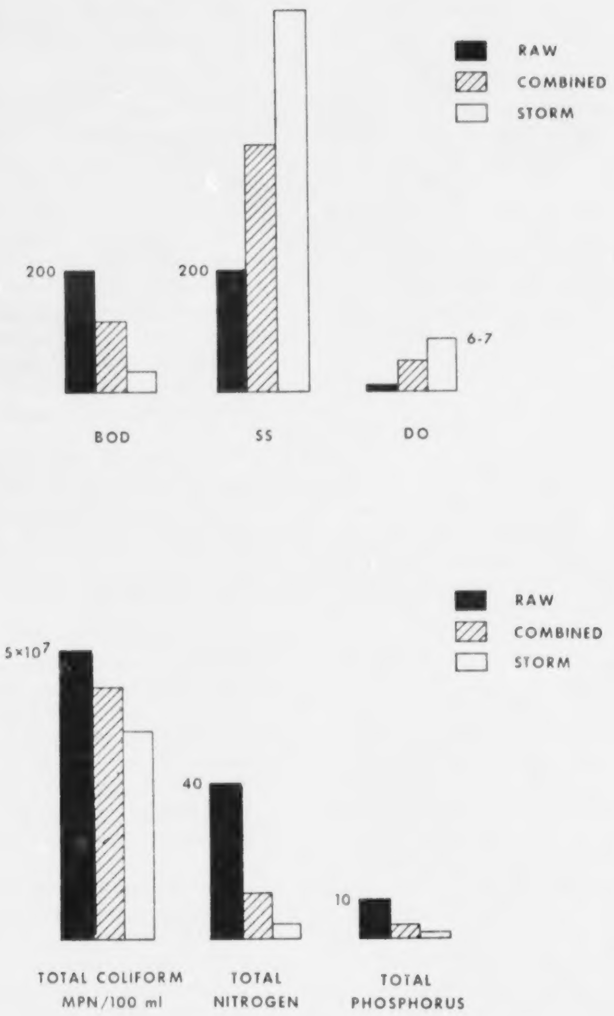

Figure 3. Representative Strengths of Wastewaters (Flow Weighted Means in mg/l)

"When a city takes a bath, what
do you do with the dirty water?"

Four types of discharges are involved: combined sewer overflows (CSO), storm drainage in separate systems, overflows from infiltrated sanitary sewers and unsewered runoff. Because of the inability to control the latter, it is usually forgotten. Significantly, the storm path and collection system configuration may have a pronounced influence on combined overflow quality, resulting in simultaneous discharge mixtures of sewage and runoff at different points, varying from raw to highly diluted as the system adjusts to a particular storm pattern. The problem constituents of general concern are visible matter, infectious bacteria, organics, and solids and in addition may include nutrients, heavy metals and pesticides.

Characterization

Representative Concentrations

Figure 3 gives some representative concentrations for comparison purposes. As shown the average BOD concentration in combined sewer overflow is approximately one-half the raw sanitary sewage BOD. However, storm discharges must be considered in terms of their shockloading effect due to their great magnitude. A not uncommon rainfall intensity of 1 in./hr. will produce urban flowrates 50 to 100 times greater than the dry-weather flow (DWF) from the same area. Even separate storm wastewaters are significant sources of pollution, "typically" characterized as having solids concentrations equal to or greater than those of untreated sanitary wastewater, and BOD concentrations approximately equal to those of secondary effluent. Bacterial contamination of separate storm wastewaters is typically 2 to 4 orders of magnitude less than that of untreated sanitary wastewaters. Significantly, however, it is 2 to 4 orders of magnitude greater than concentrations considered safe for water contact activities.

Microbiological studies of both sanitary sewage and storm runoff have shown a consistently high recovery of both pathogenic and indicator organisms (160). The most concentrated pathogens were Pseudomonas aeruginosa and Staphylococcus aureus at levels ranging from 10^3 to 10^5 and from 10^0 to 10^3/100 ml, respectively. Salmonella and enteroviruses, though frequently isolated were found at levels of only 10^0 to 10^4/10 liters of urban runoff. This strongly indicates that all types of urban runoff, in general, can be hazardous to health.

Past characterization studies for storm flow provide a data base for pollutant source accumulation, and hydraulic and pollutant loads (2, 20, 34, 35, 41, 47, 51, 53, 54, 59, 60, 63, 65, 67, 73, 81, 82, 83, 88, 102, 123, 124, 127, 128, 143, 149). A computerized data base and retrieval system has been developed for urban runoff (P-49). The data base contains screened and reasonably accurate data that is intended for model verification and future study area data synthesis -- especially useful to 201 and 208 (Section 208, PL 92-500) planning agencies.

Besides the more generalized characterization studies, specific studies have been carried out for deicing salt (67, 109, 86), and sediment/erosion (129).

Representative Loads

From 40% to 80% of the total annual organic loading entering receiving waters from a city is caused by sources other than the treatment plant (R-1). Assuming treatment plants are operating properly, during a single storm event, from 94% to 99% of the organic load and almost all settleable solids are attributed to wet-weather flow (WWF) sources (R-1).

The runoff of toxic pollutants, particularly heavy metals, is also high -- considerably higher than typical industrial discharges. For example, New York Harbor receives metals from treatment plant effluents; discharges from combined sewer overflows and separate storm sewers; and untreated wastewater included in the CSO and from sewered areas not yet served by treatment plants. As can be seen in Table 2, urban runoff is the major contributor of heavy metals to the Harbor (R-2).

Potential Impacts

Approximately one-half of the stream miles in this country are water quality

TABLE 2 - METALS DISCHARGED TO THE HARBOR FROM NEW YORK CITY SOURCES

SOURCE	Cu	Cr	Ni	Zn	Cd
Plant effluents	1,410	780	930	2,520	95
Runoff*	1,990	690	650	6,920	110
Untreated wastewater	980	570	430	1,500	60
Total weight (lb/day)	4,380	2,040	2,010	10,940	265
Weighted average concentration (mg/l)	0.25	0.12	0.11	0.62	0.015

* In reality, shockload discharges are much greater.

limited and 30% of these stream lengths are polluted to a certain degree with urban runoff. Hence, generally speaking, secondary treatment of DWF is not sufficient to produce required receiving water quality; and control of runoff pollution becomes an alternative for maintaining stream standards. Accordingly, both water quality planning and water pollution abatement programs need to be based on an analysis of the total urban pollution loads.

Until the urban stormwater situation is analyzed and efficient corrective measures taken, there may be no point to seeking higher levels of treatment efficiency in existing plants. For example:

--In Roanoke, VA domestic waste load removal was upgraded from 86% to 93%, yet there was no dramatic reduction in the BOD load (3.2 million pounds before upgrading, compared to 3.1 million pounds after) (41).

--If Durham, NC provided 100% removal of organics and suspended solids from the raw municipal waste on an annual basis, the total reduction of pollutants discharged to the receiving water would only be 59% of the ultimate BOD, and 5% of the suspended solids (112).

These examples are for separate systems. Communities with combined systems offer a potentially greater pollutional impact since additional loads come from domestic wastewaters, dry-weather sediment wash-out, and more impervious and populated lands.

Receiving Water Quality Impacts

For the aforementioned Durham study it was found that during storm flows, dissolved oxygen content of the receiving watercourse was independent of the degree of treatment of municipal wastes beyond secondary treatment. Oxygen sag estimates were unchanged even if the secondary plant was assumed upgraded to zero discharge, and stormwater discharges governed the oxygen sag 20 percent of the time.

There is an R&D study (P-68) in the Milwaukee area to determine water quality impacts from wet-weather discharges. This study is being conducted in conjunction with a Step 1 construction grant (Section 201 of PL 92-500) for the evaluation of combined sewer overflow pollution and control; and will provide the necessary "receiving water impact" basis for these evaluations.

Early results from direct receiving water sampling in the Milwaukee River provide strong evidence of CSO impacts on intensifying D.O. sag and increasing fecal coliform concentration. Figure 4 represents D.O. analyses for the Wells Street sampling station that lies at the downstream portion of the combined sewer area. Samples were collected at three hour intervals during 72 hours of dry weather during June 1975, averaged for the stream cross-section, and followed approximately nine days of antecedent dry weather. D.O. values hovered around 6 to 8 mg/l.

Figure 5 is for the same Wells Street location representing data from six days of monitoring following a 0.26 inch rainfall on October 14-15, 1975. Continuous monitoring at the site showed D.O. levels between 5.0 and 7.8 mg/l for the three days prior to rainfall. (The lag between the end of the storm and beginning of data acquisition was due to equipment malfunction). The graph indicates a highly significant D.O. sag to zero mg/l and six days after the storm required for recovery.

Adverse combined sewer overflow effects on fecal coliform concentrations in the Milwaukee River in the proximity of Lake Michigan were also deciphered. Figures 6 and 7 depict fecal coliform in the Milwaukee River during the same dry- and wet-weather monitoring periods as in Figures 4 and 5, respectively. Additionally, Figures 6 and 7 contain the Brown Deer Road monitoring site which is well above the intensely urbanized combined sewer overflow area. There is nearly a two log increase in enteric microorganisms downstream in the CSO area after wet-weather discharges indicating a potential health hazard for the nearby Lake beach fronts. Brown Deer Road showed no significant difference in fecal coliform concentration.

Due to Health Department findings, shell fishing must cease in Narragansett Bay in the vicinity of the Providence, RI overflows for periods of seven and ten days following rainfalls of one-half and one inch, respectively.

Other studies (P-15, 157, R-3) based on mass balance effects of urban runoff in receiving waters have reinforced these findings. A substantial additional effort is planned to document the receiving water impact of urban wet-weather discharges.

Erosion/Sediment Impacts

Erosion-sedimentation causes the stripping of land, filling of surface waters, and water pollution. Urbanization causes accelerated erosion, raising sediment yields two to three orders of magnitude from 10^2 - 10^3 tons/sq mi/yr to 10^4 - 10^5 tons/sq mi/yr (164). At the present national rate of urbanization, i.e., 4,000 ac/day, erosion/sedimentation must be recognized as a major environmental problem.

Nationwide Cost Assessment

Sewer Separation -- The concept of constructing new sanitary sewers to replace existing combined sewers has largely been abandoned for pollution control due to enormous costs, limited abatement effectiveness, inconvenience to the public, and extended time for implementation. The use of alternate measures for combined sewer overflow control could reduce costs to about one-third the cost for separation (2, 102). It is emphasized that sewer separation would not cope with the runoff pollution load.

High Costs Implied -- However, even in alternate approaches high costs have been implied. The 1974 Needs Survey (R-4), the 1967 EPA survey by the American Public Works Association (2), and the 1975 National Commission on Water Quality (NCWQ) Report (R-5), identified national costs for abating combined sewer overflow pollution at $26 billion, or approximately one-fourth of the total for municipal sewage control. The cost of abating separate stormwater pollution was estimated at $235 billion by the Needs Survey and $173 billion (for 75% BOD reduction) by another NCWQ report (R-6).

There must be a more accurate assessment of the problem both nationwide and regional to provide the necessary foundation for policy and law making, and firmer pollution abatement targets -- realistically, can a job be done for the money allotted?

New R&D Estimates Imply Lower Costs -- The recently completed Nationwide Assessment Report (157) has attempted to more accurately assess these national cost estimates by reflecting a more logical consideration of such items as: climate, land usage, and degree of urbanization; pollution abatement of storm flow only and not separate, conventional flood control; appropriate design flows; and the benefits of optimized coordinated systems of smaller storage-treatment units not taken into consideration in earlier estimates. The resultant national cost for combined sewer overflow and separate stormwater pollution control was $23 billion at 75% annual BOD removal (Curve A, Figure 8)(157). It is estimated that the incremental costs for combined sewer overflow pollution abatement alone would be $9 billion (Curve D, Figure 8).

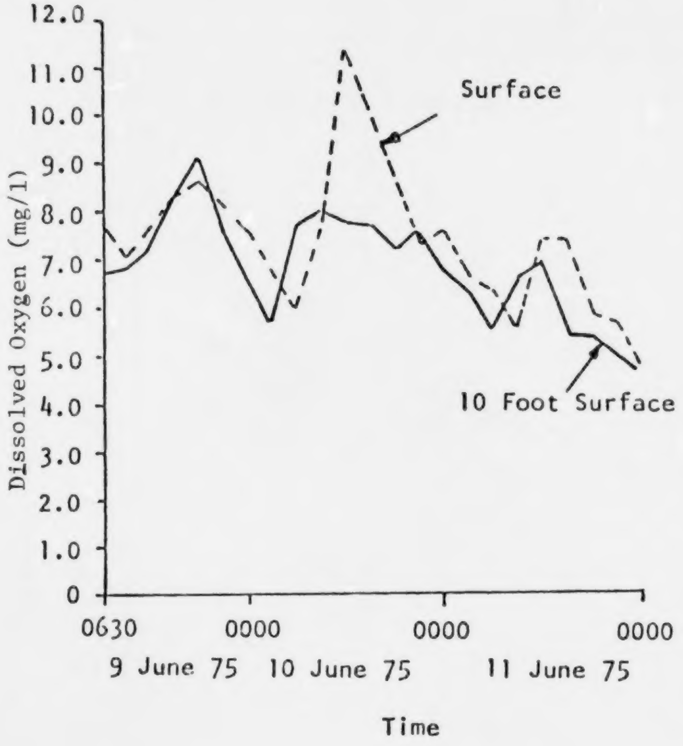

Figure 4. Dry Weather

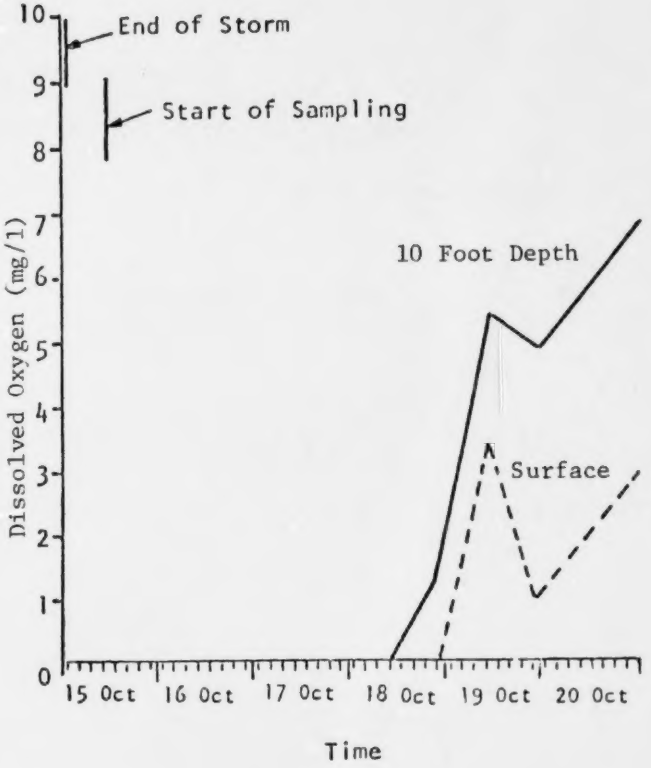

Figure 5. Wet Weather

Dissolved Oxygen Concentration, Wells Street
Milwaukee River, Milwaukee, WI

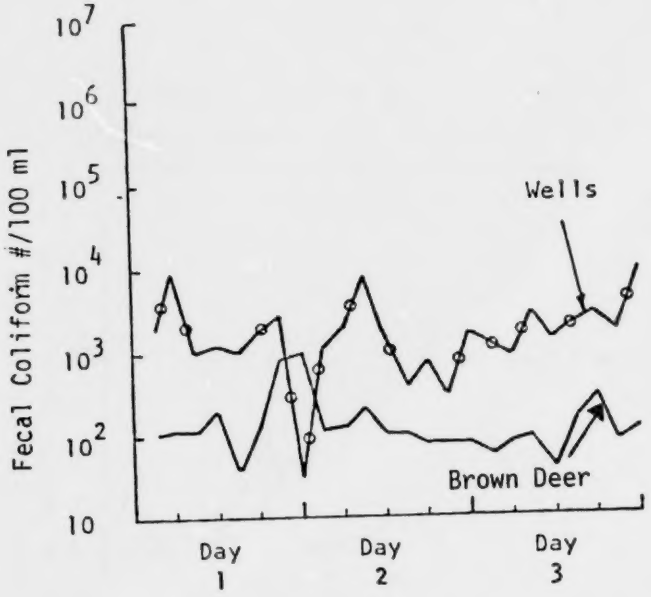

Figure 6. Dry Weather

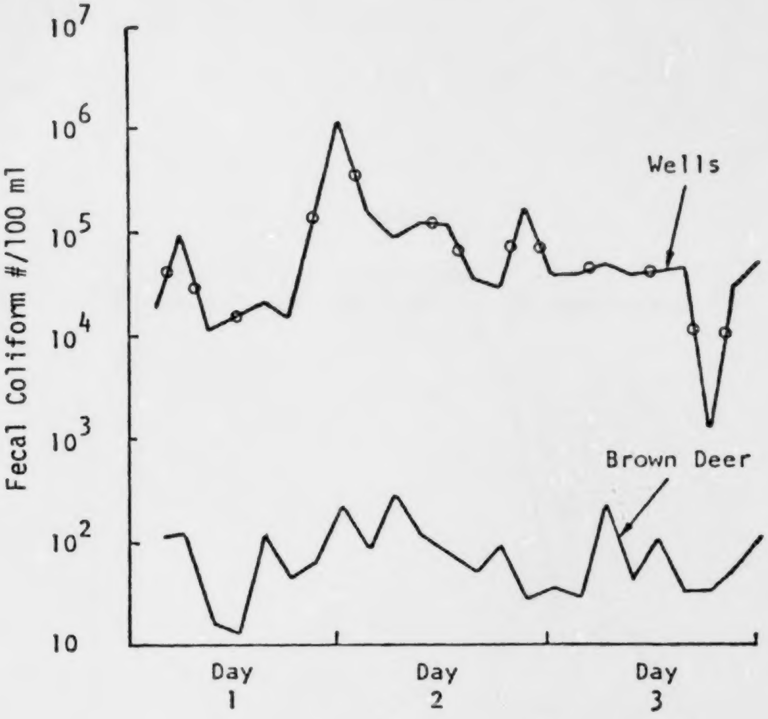

Figure 7. Wet Weather

Fecal Coliform Concentrations, Wells Street (CSO Area)
& Brown Deer Road (Separate Drainage Area)
Milwaukee River, Milwaukee, WI

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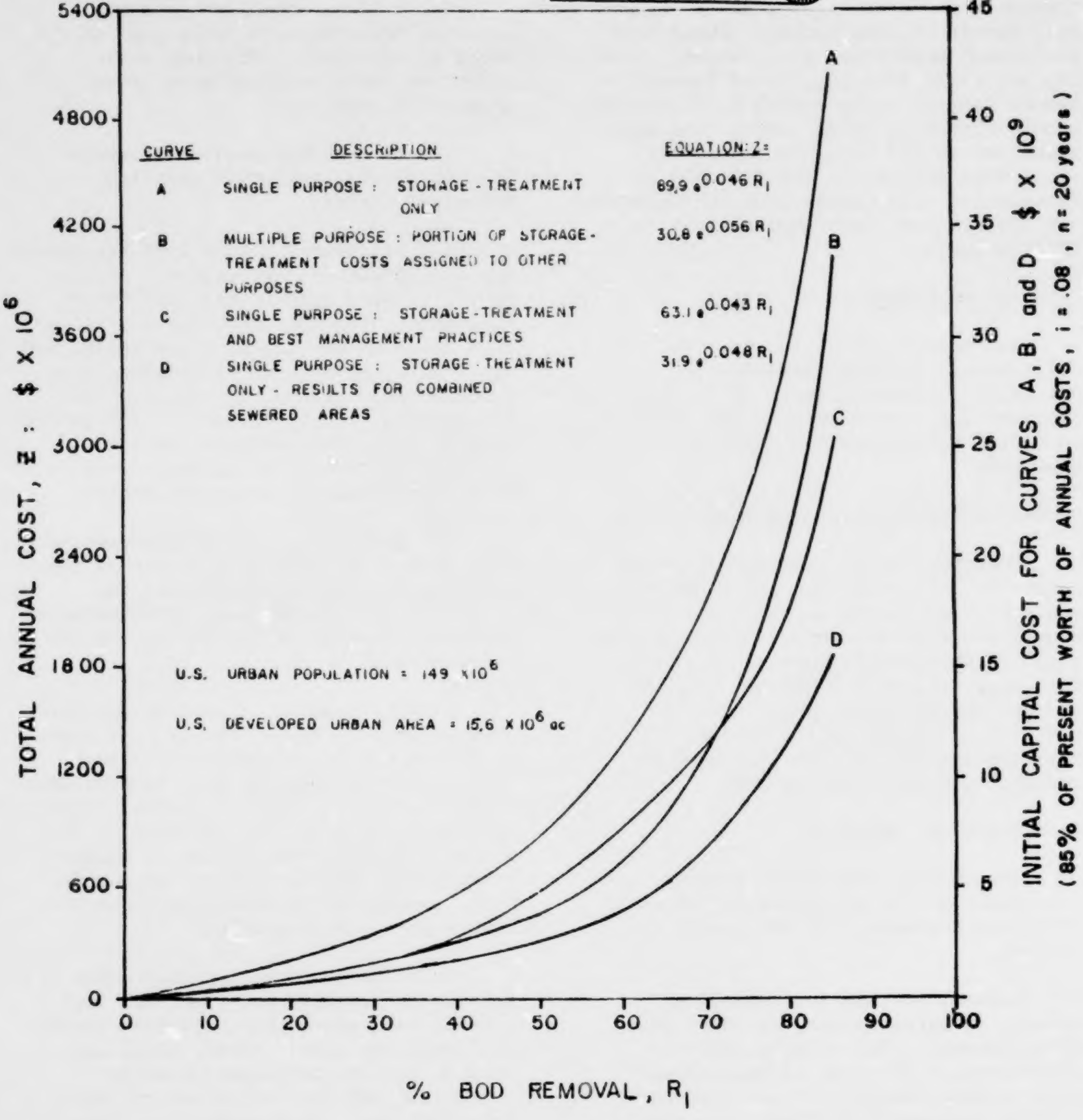


Figure 8. Single Purpose and Multiple Purpose Stormwater Pollution Control Costs for US

Additional national cost reductions were shown by the multi-purpose coordinated use of wet and dry weather flow treatment facilities, and storm flow storage facilities used as dual sedimentation-treatment processes (see Curve B, Figure 8). Within certain control levels best management practices, e.g., street cleaning and sewer flushing, could further reduce control costs (Curve C, Figure 8). When compared to prior studies the major reduction in the national figure for stormwater control is attributable to discounting storm sewer line construction (at $84 billion) and flood control (at $73 billion).

Solution Methodology

The second area under Problem Definition, "Solution Methodology" naturally followed initial "Characterization" for providing a uniform and necessary background for the user community.

More Accurate Problem Assessment

Considering the limitations in the presently available data base, the first and most fundamental approach should be a more accurate assessment of the problem. Ideally, this should involve acquiring data on a city-wide basis for both DWF and wet-weather flow (WWF) including upstream-downstream pollutants mass balances and the effects of the waste loads on the receiving waters.

Cost-Effective Approach

Integrated with a more accurate assessment is the consideration of cost-effective approaches to WWF pollution control.

Present abatement alternatives exhibit an extraordinary range of cost-effectiveness. For example, cost-effectiveness in terms of dollars/lb of pollutant removed for an alternative such as storage plus primary treatment, varies over a range of 75:1, depending on such factors as location and land costs, type and condition of sewerage systems, pollution loads, and type of storage configuration. This very high cost-effectiveness variability demonstrates the irrationality of any attempt to prescribe uniform national standards for the technology of total urban load abatement as opposed to requiring site-specific studies.

There is an excellent opportunity to bring down the high costs implied for storm flow control. The most cost-effective solution methodology must thoroughly consider:

(1) Wet-weather pollution impacts in lieu of blindly upgrading existing municipal plants.

(2) Structural versus land management and non-structural techniques. Studies have indicated that it may be cheaper to remove pollutants from the source by such measures as street, catch basin, and sewer cleaning than by eliminating them by downstream treatment. Certain land use, zoning, and construction site erosion control practices are other ways of alleviating the solids burden to the receiving stream or treatment plant;

(3) Integrating dry and wet-weather flow systems to make maximum use of the existing sewerage system during wet conditions and maximum use of wet-weather control/treatment facilities during dry weather; and

(4) The segment or bend on the percent pollutant control versus cost curve (see Figure 8 for example) where cost differences increase at much higher rates than pollutant control increases. This phenomenon is caused by the need to size storage-treatment facilities at disproportionately greater capacities for the less frequent storm events required for higher pollutant controls.

Until two important philosophies are allowed to prevail, the high cost implications for wet-weather pollution abatement will continue. First, flood and erosion control technology must be integrated with pollution control technology so that the retention and drainage facilities required for flood and erosion control can be simultaneously designed for integrated dual-benefits of pollution control. Second, if land management and non-structural techniques are maximized and integrated, there will be less to pay for the extraction of pollutants from storm flows in the potentially more costly downstream plants.

Example Solution Methodology

It is worthwhile to discuss a hypothetical example of a cost-effective solution methodology. Figure 9 represents one such approach. This case is for D.O. Actual studies should include other parameters and should represent at least one year of continuous data (at a minimum rainfall data). By this analysis a truer cost-effectiveness comparison can be made based on total time of receiving water impacts and associated abatement costs. For example, if a 5 mg/l D.O. is desired in the receiving water 75% of the time as a standard, an advanced form of wet-weather treatment or primary wet-weather treatment integrated with land management is required. The latter is the most cost-effective at $3 million. This or similar methodologies (157, Chapter VII) can help set cost-effective standards as well as select alternatives.

There is a critical lacking of meaningful water quality standards -- especially for storm flow transient effects. This limitation forces the use of (1) existing criteria not well backed up by ecological receiving water effects, or (2) arbitrary percent control of combined sewer overflow or storm discharges. A critical need exists for the technology development sectors to join together with the receiving water ecology sectors to define and establish wet-weather receiving water effects. It is felt that the present state-of-the-art is advanced far enough to generate approximate land runoff pollutant loadings from different control options and subsequent receiving water pollutant concentration. By filling the important gap of an adequate set of receiving water quality standards, the necessary foundation tools will be available for a true cost-effective solution methodology.

Administrative Problems

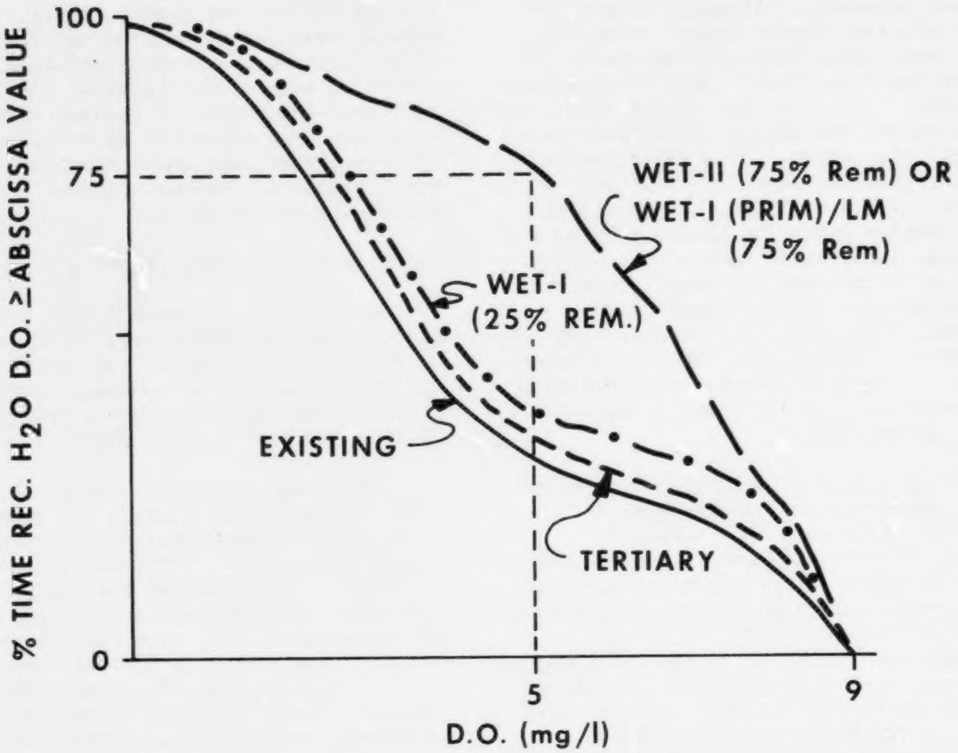
There are basic problems in administration that must be overcome. In the United States autonomous Federal and local agencies and professions involved in flood and erosion control, pollution control, and land management and environmental planning must be integrated at both the planning and operation levels. Multi-agency grant coverage must be adequate to stimulate such an approach. For example, EPA would have to join with the Army Corp of Engineers, Soil Conservation Service, Department of Transportation, and perhaps other Federal agencies as well as departments of pollution control, sanitation, planning and flood control at the local level. EPA's present policy of funding construction will also need expansion to cover cost-effective land management and non-structural techniques promulgated by its planning grant approach.

Solution Methodology: Products

Highlighted solution methodology products are the often referenced eight city studies (41, 51, 53, 54, 49, 60, 65, 83) which involved an economic comparison of pollution control alternatives for both dry and wet weather flow.

The state-of-the-art (SOTA) text on urban stormwater management and technology (102) is considered an excellent program milestone and guide for planners and engineers. It organizes and presents more than 100 completed Program projects as of December 1973. The text is presently being updated and will include comprehensive guidelines for total city-wide, wet-weather pollution control planning and countermeasure selection (P-5). Other in-house Program documents (111, R-6a, R-6b, R-6c, R-6d, R-6e, R-6f, R-6g, R-6h, R-6i) must also be included in this category.

A film is available covering the entire Program, and in particular full-scale control technologies (R-7). Program seminar proceedings (6, 40, 96) with themes of "design, operation, and costs" have been published. Urban runoff seminar proceedings for 208 planning agencies (140a) are also available. Separate engineering manuals are available for urban storm flowrate and volume determination (140, 123), storm sewer design (71), and conducting urban stormwater pollution and control studies (145). SOTAs on storm flow measuring (130) and sampling (87, 133) have also been published. All these documents are valuable references for the planning and implementing of urban stormwater studies for PL 92-500, 201, Step 1, and 208 grants.



CONTROL ALTERNATIVES	% BOD REMOVAL		COST ($\$\times10^6$)
	DRY WEATHER	WET WEATHER	
EXISTING	85	0	–
TERTIARY	95	0	6
WET-I (PRIMARY)	85	25	1
WET-II (ADV)	85	75	6
WET-I/LAND MGMT.	85	75	3

Figure 9. Example Solution Methodology

In the area of "unit cost information" a manual (156) is being published which contains summary unit cost graphs on construction and operation of the basic urban stormwater storage and treatment devices. An example on storage facility construction costs is presented in Figure 10. Additional cost information and equations can be obtained from the abovementioned text on urban stormwater management and technology (102), the SWMM user's manual (116), the nationwide stormwater assessment document (157), and the manual for preliminary (level I) stormwater control screening (153).

Other manuals are available for deicing pollution (100, 104) and erosion control (68, 70, 90, 92, 168, 169). The SOTA document on size and settling velocity characteristics of particles in storm and sanitary water (115) is important because it offers information for physical treatability of suspended solids and anticipated settlement in receiving waters. More information of this nature, along with the availability of pollutants with the suspended solids, is needed. These, along with the aforementioned solution methodology documents, are or should be serving for 201 and 208 studies.

Looking to the near future a city-wide demonstration (P-15) of a multifaceted approach methodology is nearing completion in Rochester, NY. The product from this study will serve as an example for other cities.

There is also an endeavor to study direct receiving water impacts along with verification of a water quality model. This task will serve as an important demonstration by lending credence to the implications of storm flow impacts. The previously discussed Milwaukee project (P-68) covers this objective. Other demonstration sites are being sought by the Program. Receiving water impacts have been included in an ongoing project in Lancaster, PA (P-4) and additional non-EPA funds to conduct a receiving water impact analysis for the ongoing Rochester project (P-15) have been secured.

USER ASSISTANCE TOOLS

The User Assistance Tools are divided into "Instrumentation" and "Simulation Models."

Instrumentation

The qualitative and quantitative measurement of storm overflows is essential for planning, process design, control, evaluation, and enforcement. "Urban intelligence systems" require real-time data from rapid remote sensors in order to achieve remote control of a sewerage network. Sampling devices do not provide representative aliquots, and in-line measurement of suspended solids and organics is needed. Conventional rate-of-flow meters have been developed mainly for relatively steady-state irrigational streams and sanitary flows and not for the highly varying surges encountered in storm and combined sewers. A schematic of instrumentation development by the Program is shown on Figure 11.

The electromagnetic (P-45), ultrasound (150), and passive sound (139) flowmeters have been developed to overcome these adverse storm flow conditions (which require dual pressure-gravity measurement of unsteady flows by non-intrusive instrumentation). Further demonstration of the electromagnetic and passive sound flowmeters will take place shortly. Passive sound instruments offer the additional benefit of extremely low power requirements rendering them amenable to installation at remote overflow locations (where power may not exist) and integration into city-wide, in-sewer, sensing, and control systems. A prototype sampler for capturing representative solids in storm flow, and overcoming storm flow adversities, has been developed and compared with conventional samplers. Favorable results have been obtained and a design manual (135) is available. Demonstration of two previously developed instantaneous, *in situ* monitoring devices for suspended solids (113) (based on the optical principle of suspended solids depolarizing polarized light) and TOC (126) were successfully conducted.

Separate SOTA reports for flow measurement (130) and sampling (87, 133) have been mentioned under problem definition. A SOTA on organic analyzers (110) is also available. Because storm flow conditions are extremely adverse, the manuals and instruments developed for the Program in this area are useful for the monitoring of all types of waste flows.

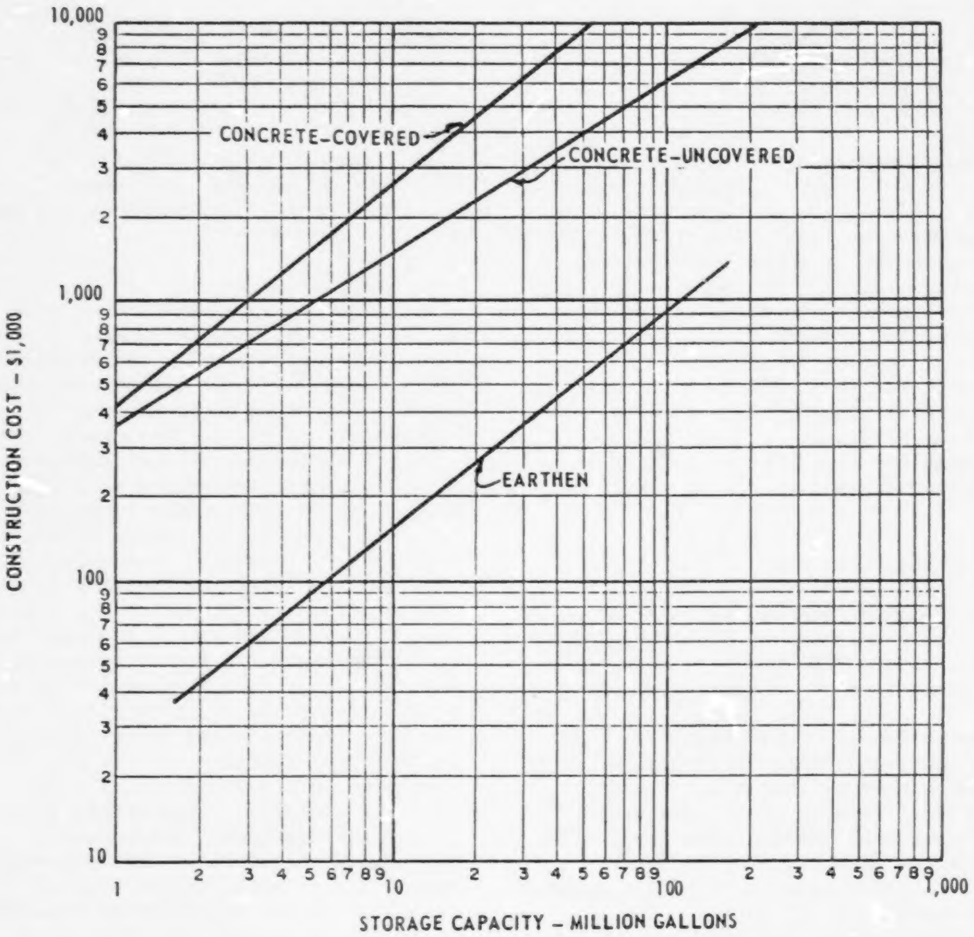


Figure 10. Construction Cost Example: Storage Facilities

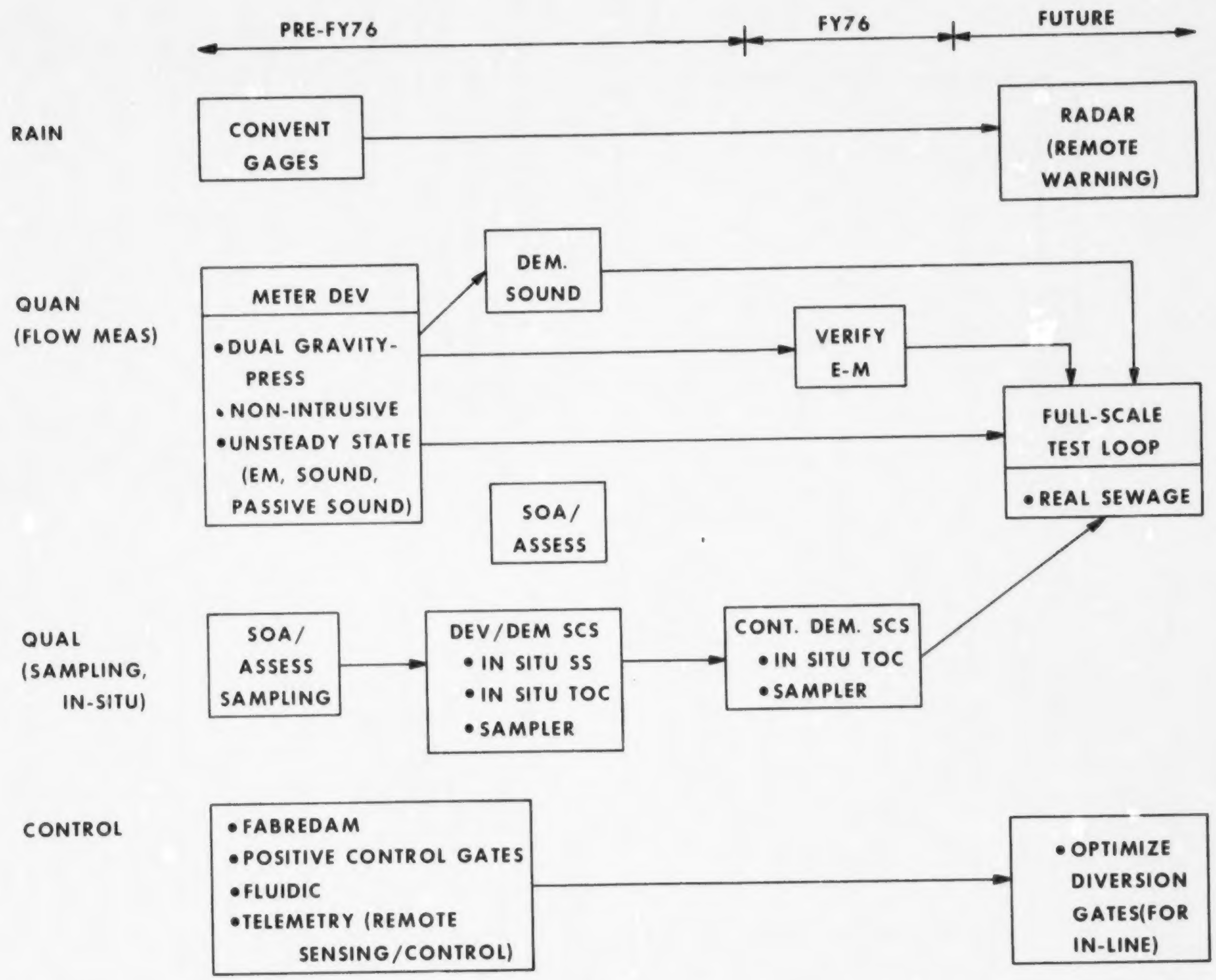


Figure 11. Instrumentation for Total System Management

Remote raingaging by radar is being considered for an automated combined sewer flow routing project in San Francisco (P-25).

Instrumentation: Products

An instrumentation product summary is listed on Table 3.

TABLE 3 - INSTRUMENTATION: PRODUCTS

Flow Measuring Devices Development

  - Electromagnetic (open-channel and press flow) (P-45)
  - Ultra-sound (150)
  - Passive sound (139)

Sampler Development (135)

In situ suspended solids monitor development (113)

In situ TOC monitoring system development (126)

SOTA/Assessment reports

  - Sampling (133)
  - Flow measuring (130)
  - Organics monitoring (110)

Simultation Models

Math models are needed to predict complex dynamic responses to variable and stochastic climatological phenomena. Models have been subcategorized into three groups: (1) *simplified* for preliminary planning, (2) *detailed* for planning and design, and (3) *operational* for supervisory control (Figure 12).

The Storm Water Management Model (SWMM) provides a detailed simulation of the quantity and quality of stormwater during a specified precipitation event. Its benefits for detailed planning and design have been demonstrated and the model is widely used. However, for many users it is too detailed; e.g., the 208 planning effort needs simplified procedures to permit preliminary screening of alternatives. Consequently, current Program thinking on urban water management analysis in general, and SWMM in particular, involves four levels of evaluation techniques ranging from simple to complex procedures than can be worked together. The major portions of all four levels have been developed (Table 4).

Planning/Design Models

Level I -- The Level I procedure as developed by the University of Florida (153) was directly derived from the previously mentioned nationwide cost assessment project (157). This assessment document already contains data on land use; drainage system types; runoff volumes and pollutant quantities; costs and cost-effective control strategies for the 248 Standard Metropolitan Statistical Areas in the Country. The information, also itemized for States and EPA regions, can be used in the early stages of problem assessment, determining national cost requirements and preliminary planning.

In Level I, a "desktop" statistical analysis procedure permits the user to estimate the quantity and quality of urban runoff in the combined, storm and unsewered portions of each urban area in his jurisdiction.

For example, under the University of Florida approach, equations such as those shown in Table 5 have been statistically developed to estimate BOD_5, SS, VS, PO_4 and

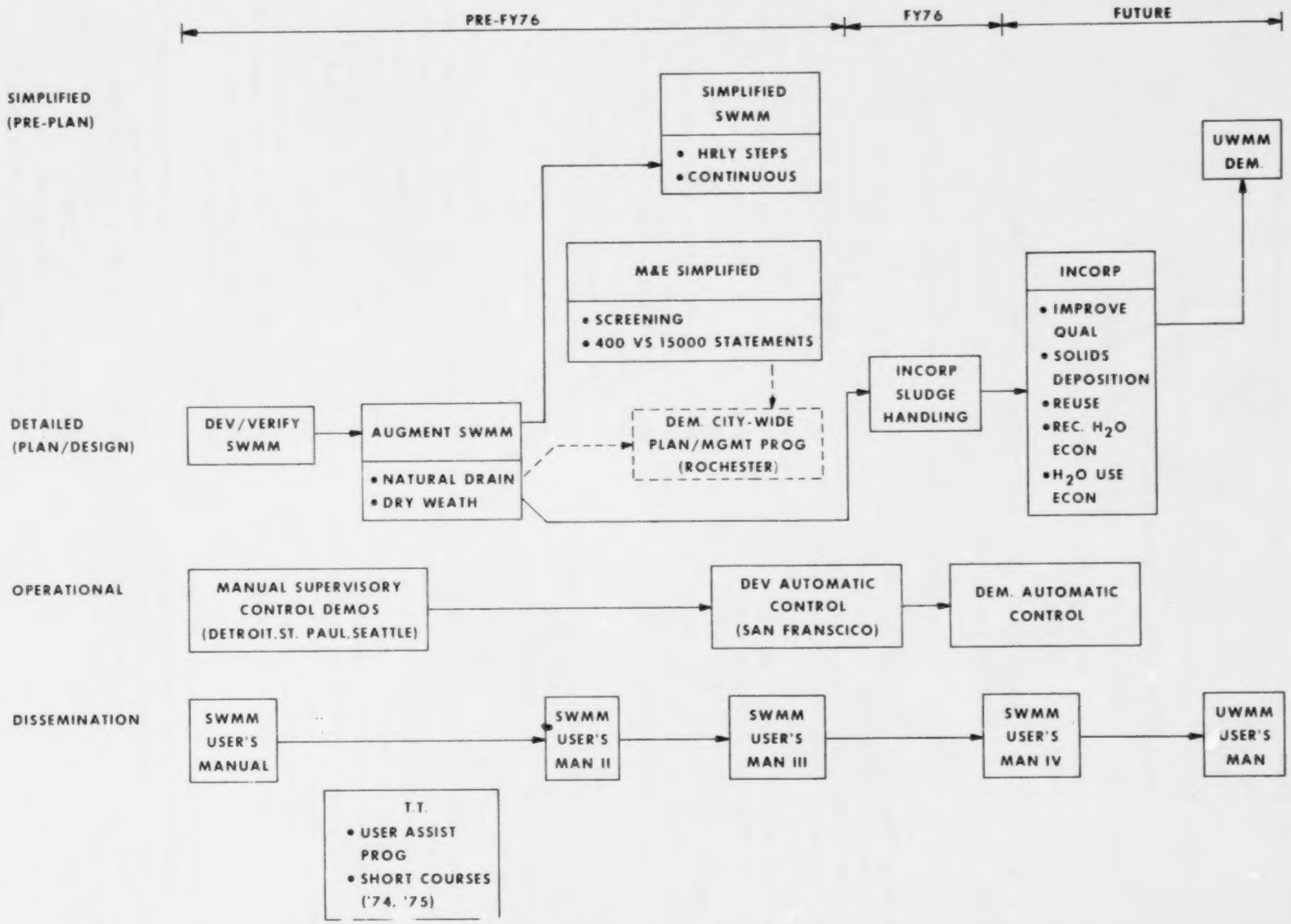


Figure 12. Simulation Models for Total System Management

TABLE 4 - LEVELS OF URBAN WATER MANAGEMENT ANALYSIS

Preliminary:	Print out information from Nationwide Assessment (157)
Level I:	Desktop - no computer, statistical analysis - UF Methodology (153) - Hydroscience methodology (R-8)
Level II:	Simplified continuous simulation model - Simplified SWMM (by M&E) (148)
Level III:	Refined continuous simulation model - Continuous SWMM (P-53) - Storm (R-9, R-10)
Level IV:	Sophisticated single event simulation model - Detailed SWMM (116, 125)

N loads as a function of land use, type of sewer system, precipitation, population density, and street sweeping frequency. The α and β terms represent normalized loading factors in lb/ac-in., tabularized as functions of land use, i and pollutant type, j, for separate and combined areas, respectively. These factors were derived from a statistical review of available stormwater pollutant loading and effluent concentration data (157).

Similarly, Table 6 gives equations for analyzing runoff for both stormwater flow prediction and DWF prediction. Here again the equations were based on a statistical analysis of available data.

A generalized method for evaluating the optimal mix of storage and treatment and its associated costs has also been developed. Also, procedures for comparing tertiary treatment with stormwater management and possible savings from integrated management of domestic wastewater, stormwater quality and stormwater quantity from combined and separate drainage areas, are available.

The Hydroscience approach offers another procedure for assessing urban pollutant sources, loadings, and control. Both approaches, available in the form of user's manuals, are being published (153, R-8).

Level II -- Level II involves a simplified continuous model for planning and preliminary sizing of facilities. The model can run on daily time steps to screen the entire history of rainfall records or hourly time steps to screen the worst two years. It is inexpensive to set up and use, flexible enough to be applicable to a variety of system configurations, and accurate even though only very moderate expenditures are made for data collection and preparation. It is especially valuable in sizing storage facilities based on storm return periods and available in-line capacity. A user's manual is available (148).

Level III -- Level III involves a more refined continuous model approach (e.g., STORM, continuous SWMM) which in addition to Level II provides for flow time routing and continuous receiving water impact analyses. The number of program statements involved here is in the order of a few thousand as compared to a few hundred for the Level II effort. A user's manual and program for STORM is available (R-9, R-10). The continuous SWMM user's manual is in preparation.

Level IV -- The aforementioned three levels essentially represent various degrees of planning efforts and the models involved are typified by relatively large time steps (hours) and long simulation times (months and years). Data requirements are kept to a minimum and their mathematical complexity is low.

Table 5. Pollutant Analysis

The following equations may be used to predict annual average loading rates as a function of land use, precipitation and population density.

Separate Areas: $M_s = \alpha(i,j) \cdot P \cdot f_2(PD_d) \cdot \gamma \frac{\text{lb}}{\text{acre-yr}}$

Combined Areas: $M_c = \beta(i,j) \cdot P \cdot f_2(PD_d) \cdot \gamma \frac{\text{lb}}{\text{acre-yr}}$

where
M = pounds of pollutant j generated per acre of land use i per year,
P = annual precipitation, inches per year,
PD_d = developed population density, persons per acre,
α, β = factors given in table below,
γ = street sweeping effectiveness factor, and
$f_2(PD_d)$ = population density function.

Land Uses: i = 1 Residential
i = 2 Commercial
i = 3 Industrial
i = 4 Other Developed, e.g., parks, cemeteries, schools (assume $PD_d = 0$)

Pollutants: j = 1 BOD_5, Total
j = 2 Suspended Solids (SS)
j = 3 Volatile Solids, Total (VS)
j = 4 Total PO_4 (as PO_4)
j = 5 Total N

Population Function: i = 1 $f_2(PD_d) = 0.142 + 0.218 \cdot PD_d^{0.54}$
i = 2,3 $f_2(PD_d) = 1.0$
i = 4 $f_2(PD_d) = 0.142$

Factors α and β for Equations: Separate factors, α, and combined factors, β, have units lb/acre-in. To convert to kg/ha-cm, multiply by 0.442.

		Pollutant, j				
	Land Use, i	1. BOD_5	2. SS	3. VS	4. PO_4	5. N
Separate Areas, α	1. Residential	0.799	16.3	9.45	0.0336	0.131
	2. Commercial	3.20	22.2	14.0	0.0757	0.296
	3. Industrial	1.21	29.1	14.3	0.0705	0.277
	4. Other	0.113	2.70	2.6	0.00994	0.0605
Combined Areas, β	1. Residential	3.29	67.2	38.9	0.139	0.540
	2. Commercial	13.2	91.8	57.9	0.312	1.22
	3. Industrial	5.00	120.0	59.2	0.291	1.14
	4. Other	0.467	11.1	10.8	0.0411	0.250

Street Sweeping: Factor γ is a function of street sweeping interval, N_s, (days):

$$\gamma = \begin{cases} N_s/20 \text{ if } 0 \leq N_s \leq 20 \text{ days} \\ 1.0 \quad \text{if } N_s > 20 \text{ days} \end{cases}$$

TABLE 6 - RUNOFF ANALYSIS

Stormwater Flow Prediction

$$AR = (0.15 + 0.75\ I/100\ P - 5.234\ (DS)^{0.5957}$$

where AR = Annual Runoff, in/yr

$$I = 9.6\ PD_d^{(0.573-0.0391\ \log_{10} PD_d)}$$

where I = Imperviousness, Percent and

PD_d = Population Density in Developed Portion of the Urbanized Area, Persons/Acre

P = Annual Precipitation, in/yr and

$DS = 0.25-0.1875\ (I/100)\quad 0 \leq I \leq 100$

where DS = Depression Storage, in. $(0.005 \leq DS \leq 0.30)$

Dry Weather Flow Prediction

$$DWF = 1.34\ PD_d$$

where DWF = Annual Dry-Weather Flow, in/yr, and

PD_d = Developed Population Density, Persons/Acre

Design models on the other hand are oriented toward the detailed simulation of a single storm event. They provide a complete description of flow and pollutant routing from the point of rainfall through the entire urban runoff system and into the receiving waters. Such models may be used for predictions of flows and concentrations anywhere in the rainfall-runoff system and can illustrate the detailed and exact manner in which abatement procedures or design options affect them. At such, these models are a highly useful tool for determining least-cost abatement procedures for both quantity and quality problems in urban areas. They are typified by short time steps (minutes) and short simulation times (hours). Data requirements are usually very extensive. The EPA SWMM is such a model. SWMM user's manuals and other pertinent references that were revised at a critical junctures are available (42, 43, 44, 45, 116, 120). Eventually is is hoped than SWMM can be expanded into an Urban Water Management Model which integrates both dry- and wet-weather flow analyses including sludge handling capabilities. This is emphasized in the Program report on future direction of the modeling development (136).

Operational Models

Operational models are used to produce actual control decisions during a storm event. Rainfall is entered from telemetered stations and the model is used to predict system responses a short time into the future. Various control options may then be employed, e.g., in-system storage, diversions, regulator settings. The Program has demonstrated supervisory control models in Detroit (118), Minneapolis-St. Paul (19), and Seattle (29, 98); and has recently started on a project in San Francisco (P-25) taking advantage of a $100 million construction grant, to develop a fully automated operational model which includes rainfall prediction.

Simulation Models: Products

Other simulation model products include demonstration of a dissemination and user assistance capability (122) and development of a short course and course manual (125, P-51) for stormwater management model application. Of particular note is the SOTA assessment document on 18 available mathematical models for storm and combined sewer management (141).

TABLE 7 - SIMULATION MODELS: PRODUCTS

- Development of a computer model (SWMM) for storm water management (42, 43, 44, 45).
- Updated and refined user's manual modifying and improving SWMM (116).
- Demonstration of a stormwater management model dissemination and user assistance capability (122).
- User's manual for "desktop calculation" procedure for preliminary stormwater management planning (153).
- User's manual for simplified model application for preliminary stormwater management planning (148).
- Course manual and seminar for stormwater management model application (125).
- Assessment of mathematical models for storm and combined sewer management (141).
- Refine and augment the capabilities of SWMM and develop decision-making capabilities (120).
- Evaluation of available runoff prediction methods for storm flowrate and volume determination (140).

The document presents a summary of the objectives, advantages and limitations of each model along with a side-by-side comparison to aid in assessing the applicability of a model for a particular purpose. Table 7 summarizes simulation model products.

MANAGEMENT ALTERNATIVES

Wet weather flow control can be assumed to involve aspects as follows. First there is the choice as to where to attack the problem: at the source (e.g., the street, gutters, and catchment areas) by land management, in the collection system, or off-line by storage. Pollutants can be removed by treatment and by employing complex or integrated systems which combine variations of control and treatment including the dual-use of dry-weather facilities. Second, there is the choice of how much control or degree of treatment to introduce. Thirdly, there is the impact assessment, public exposure, and priority ranking with other needs. The proper management alternatives can only be made after a cost-effective analysis involving goals; values; and hydrologic-physical system evaluations, generally assisted by mathematical model simulations, pilot-scale trials, and new technology transfer.

Land Management

Land Management includes all measures for reducing urban and construction site stormwater runoff and pollutants before they enter the downstream drainage system (Figure 13). On-site measures include structural, semi-structural and non-structural techniques that affect both the quantity and quality of runoff.

Careful consideration must be given to land use planning since urbanization accelerates hydrograph and pollutograph peaks and total loads by creating impervious surfaces for pollutants and water to run off from. This causes excessive water pollution, erosion, sedimentation and flooding. Discreet selection of land management techniques can reduce drainage and other downstream control costs associated with these problems.

Until two important philosophies prevail, the high cost implications for wet-weather pollution abatement will continue. Established flood and erosion control technology must be integrated with pollution control technology so that the retention and drainage facilities and other non-structural management techniques required for flood and erosion control can be simultaneously designed for pollution control.

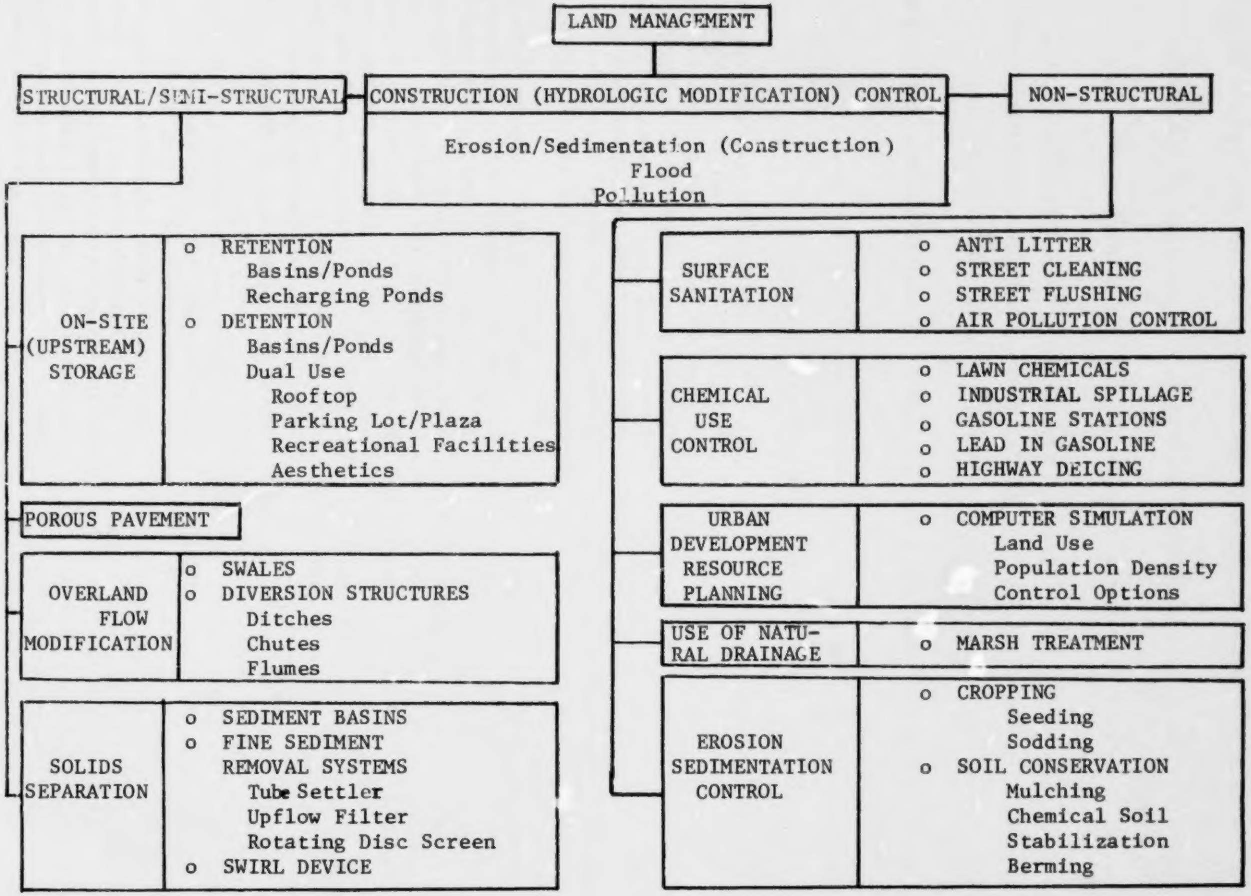


Figure 13. Land Management

Structural/Semi-Structural Control

Structural and semi-structural control measures require physical modifications in a construction or urbanizing area and includes such techniques as: on-site storage, porous pavement, overland flow modifications and solids separation.

On-Site (Upstream) Storage -- On-site or upstream storage refers to detention (short term) or retention (long term) of runoff prior to its entry into a drainage system. Simple ponding techniques are utilized on open areas where stormwater can be accumulated without damage or interference to essential activities. Oftentimes, on-site storage does or can be designed to provide for the dual or multi-benefits of aesthetics, recreation, recharge, irrigation, or other uses. For example, in Long Island, NY, groundwater supplies are being replenished by retention-recharge. The dual benefit of recharging is stressed because urbanization depletes groundwater supplies; however, potential groundwater pollution must also be considered.

Successful variations of detention that take advantage of facilities primarily used for other purposes are ponding on parking lots, plazas, recreation and park areas; and ponding on roof tops. The fundamental approach is the same as for other forms of detention but low cost is implied. Dual purpose basins used for recreation and athletics when dry are also employed.

Surface ponding is the most common form of detention being used by developers. Apparent economic benefits of surface ponding for flood protection are derived from the savings over a conventional sewer project. Several surface ponding sites are listed in Table 8 where a cost comparison is made between a drainage system using surface ponds to decrease peak flows and a conventional storm sewer system. It is important to note that pollution and erosion control benefits of the basins are not included in this comparison.

Porous Pavement -- Another approach to stormwater management is the use of an open graded asphalt-concrete pavement which under pilot testing has allowed over 70 in./hr. of stormwater to flow through (Figure 14) (64). Stability, durability, and freeze-thaw tests have been positive and it is comparable in cost to conventional pavement. Long-term tests are still required to evaluate clogging resistance and the quality of water that filters through. If the soil porosity under the pavement allows free drainage there will be no water residue; however, the coarse sub-base and porous nature of the pavement can serve for ponding capacity if storm quantities exceed soil infiltration. A 4-inch pavement and 6-inch base could store 2.4 in. of runoff volume in its voids. The proven use of porous pavement can be an important tool in preserving natural drainage and decreasing downstream drainage and pollution control facility requirements. As a result of Program studies a feasibility report (64) is available. The Program is currently evaluating a porous pavement parking lot (P-16) and results of this study will be available next year.

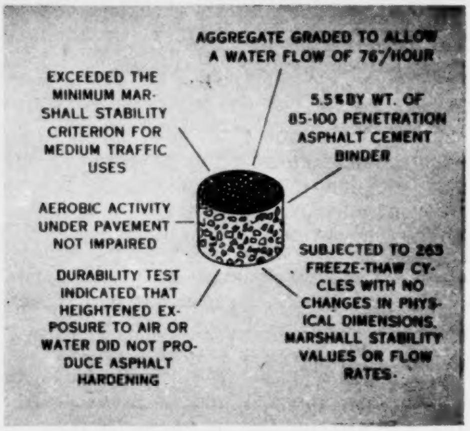

AGGREGATE GRADED TO ALLOW A WATER FLOW OF 76"/HOUR
EXCEEDED THE MINIMUM MARSHALL STABILITY CRITERION FOR MEDIUM TRAFFIC USES
5.5% BY WT. OF 85-100 PENETRATION ASPHALT CEMENT BINDER
AEROBIC ACTIVITY UNDER PAVEMENT NOT IMPAIRED
SUBJECTED TO 263 FREEZE-THAW CYCLES WITH NO CHANGES IN PHYSICAL DIMENSIONS, MARSHALL STABILITY VALUES OR FLOW RATES.
DURABILITY TEST INDICATED THAT HEIGHTENED EXPOSURE TO AIR OR WATER DID NOT PRODUCE ASPHALT HARDENING

Figure 14 - Porous Asphaltic-Concrete Features

Overland Flow Modification -- Another form of structural and semi-structural control is overland flow modification including swales and diversion structures (e.g., ditches, chutes, flumes). These modifications are usually of lower cost than subterranean sewer construction and importantly allow vegetative cover and soil infiltration to reduce runoff and pollutant loadings.

TABLE 8 - COST COMPARISON BETWEEN SURFACE PONDING TECHNIQUES AND CONVENTIONAL SEWER INSTALLATION (R-8)

Site	Description	Cost Estimate, $	
		With Surface Ponding	With Conventional Sewers
Earth City, Missouri	A planned community including permanent recreational lakes with additional capacity for storm flow	2,000,000	5,000,000
Consolidated Freightways, St. Louis, Missouri	A trucking terminal using its parking lots to detain storm flows	115,000	150,000
Ft. Campbell, Kentucky	A military installation using ponds to decrease the required drainage pipe sizes	2,000,000	3,370,000
Indian Lakes Estates, Bloomington, Illinois	A residential development using ponds and an existing small diameter drain	200,000	600,000

Solids Separation -- Sediment basins trap and store sediment from erodible areas in order to conserve land and prevent excessive siltation downstream. If designed properly, these basins can remain after construction for on-site storage. A project (P-46) is evaluating the efficiency of sediment basins.

Because a significant portion of the eroded solids may be colloidal or unsettleable and therefore cannot be treated in conventional sedimentation basins, special devices for fine-particle removal are required. An ongoing project (P-73) has developed a SOTA (163) on methods for fine-particle removal and is now undertaking the evaluation of three solids separation devices (i.e., tube settler, up-flow filter, and rotating disc screen).

The swirl concentrator has been developed for erosion control (P-3, 99) to remove settleable solids at much higher rates than sedimentation. A prototype device is presently being evaluated at a construction site (P-74).

Non-Structural

Non-structural control measures involve surface sanitation, chemical use control, urban development resource planning, use of natural drainage, and certain erosion/sedimentation control practices (Figure 13).

Surface Sanitation -- Maintaining and cleaning the urban area can have a significant impact on the quantity of pollutants washed off by stormwater. Cleanliness starts with reduction of litter and debris at the neighborhood level. Both street repair and street sweeping can further minimize the pollutants washed off. It has been estimated that street sweeping costs per ton of solids removed are about half the costs for solids removed via the sewerage system.

The effectiveness of street sweeping operations with respect to stormwater pollution has been analyzed by EPA (73, 88, 128, 157, P-49). It was found that a great portion of the overall pollution potential is associated with the fine solids fraction of the street surface

TABLE 9 - ADVANCED* STREET CLEANER POLLUTANT RECOVERY PERCENTAGE

Parameter	% Recovery
Dry Weight Solids	93
Volatile Solids	80
BOD	67
COD	84
Total PO_4-P	85
Heavy Metals	83-98

*Broom and Vacuum Combination

contaminants and that only 50 percent of the dry weight solids are picked up by conventional broom sweepers (73) as compared to 93 percent removal by more advanced techniques (128) (Table 9).

Cities clean their streets for aesthetic reasons, removing the larger particles and brushing aside the fines. Conventional sweepers are utilized and satisfy the aesthetics problem. More advanced street cleaning procedures such as a combination of sweeping and vacuuming would not only satisfy the aesthetics problem but would also attack the source of stormwater related pollution problems by removing the finer or more pollutant prone range of particles.

Further verification of the benefits of street cleaning will be carried out in an ongoing grant (p-25). Also, a desktop analysis comparing the cost-effectiveness of street cleaning and sewer flushing with downstream treatment methods is nearing completion under another study (P-73). Flushing of streets can be used to remove street contaminants effectively; however, it may necessitate more frequent catch basins and sewer cleaning. Street cleaning is estimated to cost $3 to $13/ curb mi or about $0.75/ac.

Air pollution abatement plans must also consider water pollution reduction benefits from decreased fall out.

Chemical Use Control -- One of the most overlooked measures for reducing the pollution potential from neighborhood areas is the reduction in the indiscriminate use of chemicals such as fertilizers and pesticides, and the mishandling of other materials such as oil, gasoline, and highway deicing chemicals. Aside from air pollution control, de-leaded gasoline also results in water pollution control.

The progression of studies in deicing chemical control, and resulting reports, is depicted in Figure 15. The Program's motivation from the start has been to determine the extent of environmental damages and costs associated with the use of chemical deicers so that the economic validity of alternative approaches could be assessed.

Until the Program's assessment of the problem in 1971 (67) there had been only limited research on highway deicing effects. Inquiries concerning this work indicated such an increased public awareness of the salt problems, that it seemed appropriate to firm up recommendations for alternatives to snow and ice control. A search was conducted (76) to define alternatives. The need for an accurate economic impact analysis of using deicing salt, and a requirement to identify a substance which can be applied to pavement to reduce ice adhesion was indicated. These two needs became projects which have recently been completed (138, 152). Hydrophobic substances have been identified and are being investigated, and even though material and application costs appear greater than for salt (0.20-0.25/yd^2 versus $0.03/yd^2), when considering total damage to the environment ($3 billion annually, including paved area, highway structures and vehicles) the costs are acceptable.

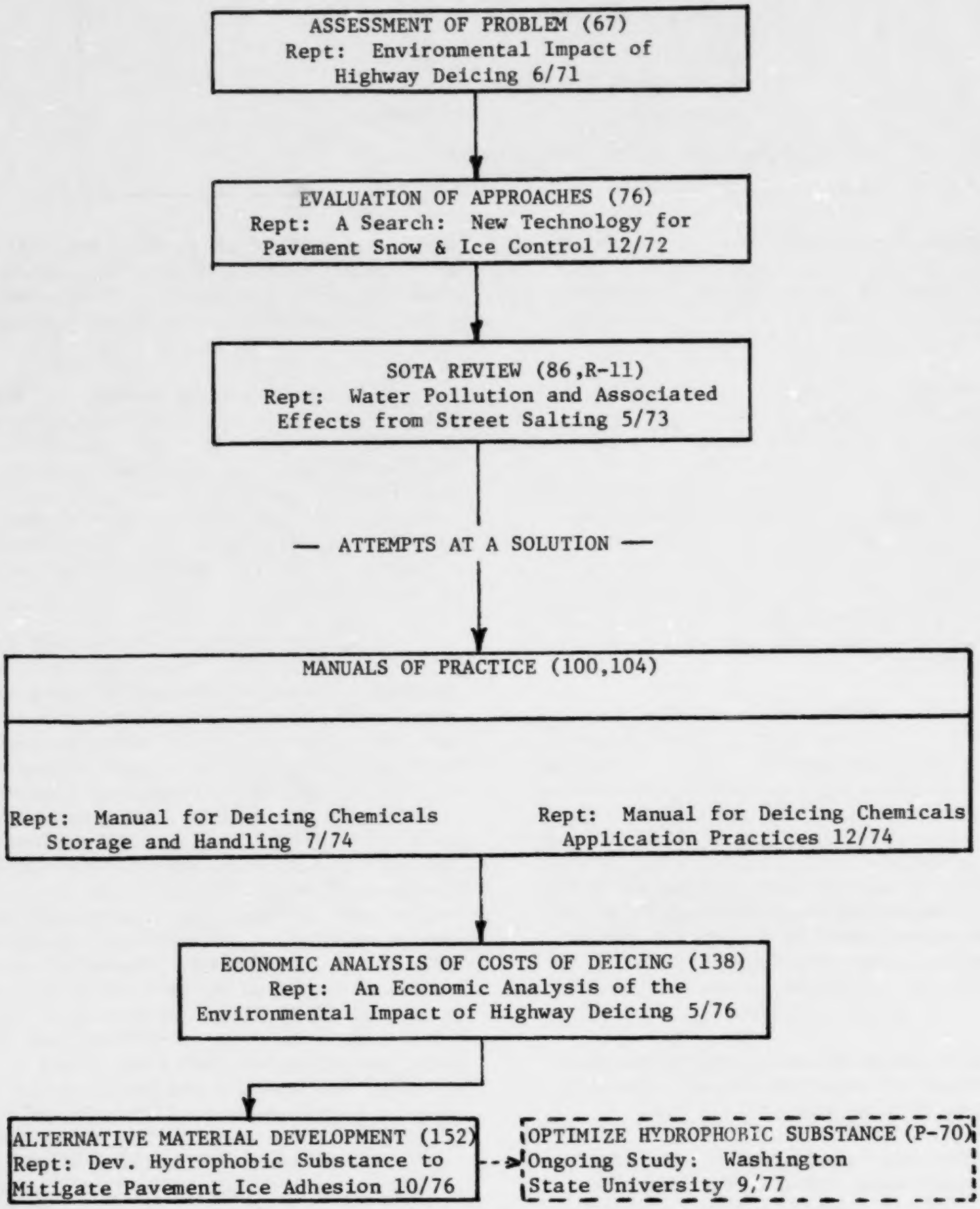

ASSESSMENT OF PROBLEM (67)
Rept: Environmental Impact of Highway Deicing 6/71
EVALUATION OF APPROACHES (76)
Rept: A Search: New Technology for Pavement Snow & Ice Control 12/72
SOTA REVIEW (86,R-11)
Rept: Water Pollution and Associated Effects from Street Salting 5/73
— ATTEMPTS AT A SOLUTION —
MANUALS OF PRACTICE (100,104)
Rept: Manual for Deicing Chemicals Storage and Handling 7/74
Rept: Manual for Deicing Chemicals Application Practices 12/74
ECONOMIC ANALYSIS OF COSTS OF DEICING (138)
Rept: An Economic Analysis of the Environmental Impact of Highway Deicing 5/76
ALTERNATIVE MATERIAL DEVELOPMENT (152)
Rept: Dev. Hydrophobic Substance to Mitigate Pavement Ice Adhesion 10/76
OPTIMIZE HYDROPHOBIC SUBSTANCE (P-70)
Ongoing Study: Washington State University 9/77

Figure 15. Deicing Chemical Control (Land Management/Non-Structural)

After the 1973 assessment of the problem (86, R-11), the Program recognized that it was not practical to ban salt since the "bare pavement" philosophy was very popular and considered by most highway authorities as the safest way for ice and snow removal. The major problems were identified with careless salt storage practices and over-application on highways, consequently, a 1974 project resulted in manuals of practice for improvement in these areas. These manuals (100, 104) were recognized as highly significant by the user community. To date, over 7000 copies have been distributed.

Urban Development Resource Planning -- The goal of urban development resource planning is to develop a microscopic management concept to prevent the problems resulting from shortsighted urbanization plans. As previously discussed, the planner must be aware of totally integrating planned urban hydrology with erosion-sedimentation and pollution control. This new breed of planner has to consider the new land development planning variables of land usage, population density and total wet and dry runoff control as they integrate to effect water pollution. Computer simulation will most likely play an important role. A simple land planning model has been developed by G. K. Young (140a, Chapter I, pp 98-121) to encompass the pertinent variables and the most effective control options based upon receiving water pollutant absorption capacity. A new project is planned to perfect this area.

Use of Natural Drainage -- The traditional urbanization process upsets the existing water balance of a site by replacing natural infiltration areas and drainage with impervious areas. The net impact is increased runoff, decreased infiltration to the groundwater and increased flowrates, all contributing to increased channel erosion and the transport of surface pollutants to the stream. Promulgating the use of natural drainage concepts will reduce drainage costs; enhance aesthetics, groundwater supplies, and flood protection; and lower pollution.

A project in Houston (P-16) focuses on how a "natural drainage system" can be integrated into a reuse scheme for recreation and aesthetics. Good land use management will allow runoff to flow through low vegetated swales and into a network of wet-weather ponds, strategically located in areas of porous soils. This sytem will cause some of the runoff to seep into the ground and retard the flow of water downstream, thus preventing floods caused by development and enhancing pollution abatement. The concept of considering urban runoff as a benefit as opposed to a wastewater, in a new community development, will be employed and evaluated.

Another project in Wayzata, MN (P-28) is using marshland for stormwater treatment. After sufficient testing it has been determined that controlled stormwater retention in the marsh resulted in better vegetative conditions which in turn enhanced stormwater nutrient removal. It was found that if the marshlands were filled in by urbanization it would have a detrimental effect on the nearby lake.

Erosion/Sedimentation Control (Non-Structural) -- Other nonstructural soil conservation practices such as cropping (seeding and sodding) and the use of mulch blankets, nettings, chemical soil stabilizers and berming may be relatively inexpensive. Two ongoing projects (P-72, P-74) are evaluating many of these low structural intensive management practices for proposed erosion control manuals.

Integrated Benefits

While the flood control benefits of all the above land management control measures are easy to see, the stormwater pollution and erosion control effects are difficult to quantify. But briefly stated, detaining or retaining flow upstream offers the opportunity for flow quiescence resulting in solids separation. It also decreases downstream drainage velocities and discharges to streams resulting in less overflow pollution, siltation and scour. Aside from causing downstream erosion, this scouring can also increase pollution loads in the scouring stream.

Erosion/Sediment Control: Products

By showing the genealogy of the products through past milestone events (Figure 16) the strategy which has guided the Program in this category can be demonstrated. The original "Guidelines for Erosion and Sediment Control Planning and Implementation" (70) are still applicable to communities initiating an urban sediment control program.

For erosion-sedimentation controls, many agencies (e.g., The Department of Transportation and Soil Conservation Service, and state and local departments) and factors must be considered and interrelated in product development and technology implementation. For example, the Soil Conservation Service has published a document with the State of Maryland entitled, "Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas" (R-12). Other states are using this document as a model ordinance. Local laws will have an important impact on any Best Management Practices proposed by EPA. Therefore, there must be close liaison between all groups.

A recently developed training program consists of an instructor's manual (168), a workbook (169), and 2762 slides with integrated audio cassettes. The program is directed to the local land developer and inspector, the excavation contractor, and the job foreman. It is designed to directly support the Maryland "Standards and Specifications for Erosion and Sediment Control in Developing Areas." As the state and local agencies move toward setting standards for control on non-point sources, the need for this type of training program becomes urgent.

Future Program plans include an evaluation of various cities' erosion control programs. This product will be the foundation for National Standards and Specifications for sediment and erosion control in developing areas and with the findings of the Urban Runoff Program will lead to the National Best Management Practice for this category.

COLLECTION SYSTEM CONTROLS

The next category, collection system control (Figure 17) pertains to those management alternatives concerned with wastewater interception and transport. These alternatives include sewer separation; improved maintenance and design of catch basins, sewers, regulators and tide gates; and remote flow monitoring and control. The emphasis, with the exception of sewer separation, is on optimum utilization of existing facilities and fully automated control. Because added use of the existing system is employed, the concepts generally involve cost-effective, low-structurally intensive control alternatives. To accomplish this an extensive and dependable intelligence system is necessary.

Catch Basins

A project is assessing the value of catch basins (P-17, 174) as they are presently designed and maintained. Optimized basin configuration, design and maintenance for removing solids before sewerage system entry have also been investigated. Evaluations showed that a catch basin contains approximately 0.18 lb-BOD_5 or the equivalent of one person's daily contribution. The utilization of catch basins can either contribute to the pollutional load or aid in reducing downstream treatment depending on their design and maintenance.

Sewers

Solids deposition in lines has always been a plague to effective maintenance. Recently, the significance of such loads as a major contributor to first flush pollution has been recognized (P-66; 140a, Chapter II, pp 62-82).

Work is being conducted on new sewer designs for low flow solids carrying velocity to alleviate sewer sedimentation and resultant first flush and premature bypassing (P-50); and also on sewer designs for added storage (P-13, 165). As a natural follow-up to Program work with a controlled test loop (13, 14), a project has just been initiated to demonstrate periodic sewer flushing during dry weather for first flush relief (P-66).

Polymers to Increase Capacity -- Research (6, 11, P-6) has shown that polymeric injection can increase flow capacity as much as 2.4 times (at a constant head). This method can be used

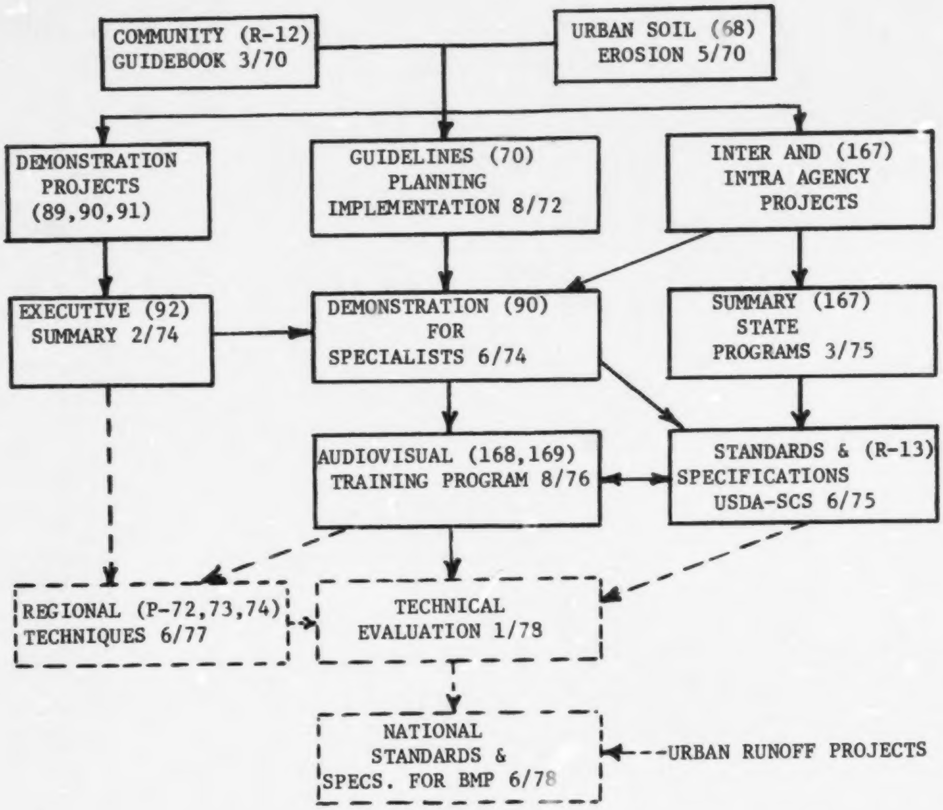

COMMUNITY (R-12) GUIDEBOOK 3/70
URBAN SOIL (68) EROSION 5/70
DEMONSTRATION PROJECTS (89,90,91)
GUIDELINES (70) PLANNING IMPLEMENTATION 8/72
INTER AND (167) INTRA AGENCY PROJECTS
EXECUTIVE (92) SUMMARY 2/74
DEMONSTRATION (90) FOR SPECIALISTS 6/74
SUMMARY (167) STATE PROGRAMS 3/75
AUDIOVISUAL (168,169) TRAINING PROGRAM 8/76
STANDARDS & (R-13) SPECIFICATIONS USDA-SCS 6/75
REGIONAL (P-72,73,74) TECHNIQUES 6/77
TECHNICAL EVALUATION 1/78
NATIONAL STANDARDS & SPECS. FOR BMP 6/78
URBAN RUNOFF PROJECTS

Figure 16. Erosion/Sedimentation Control: Products

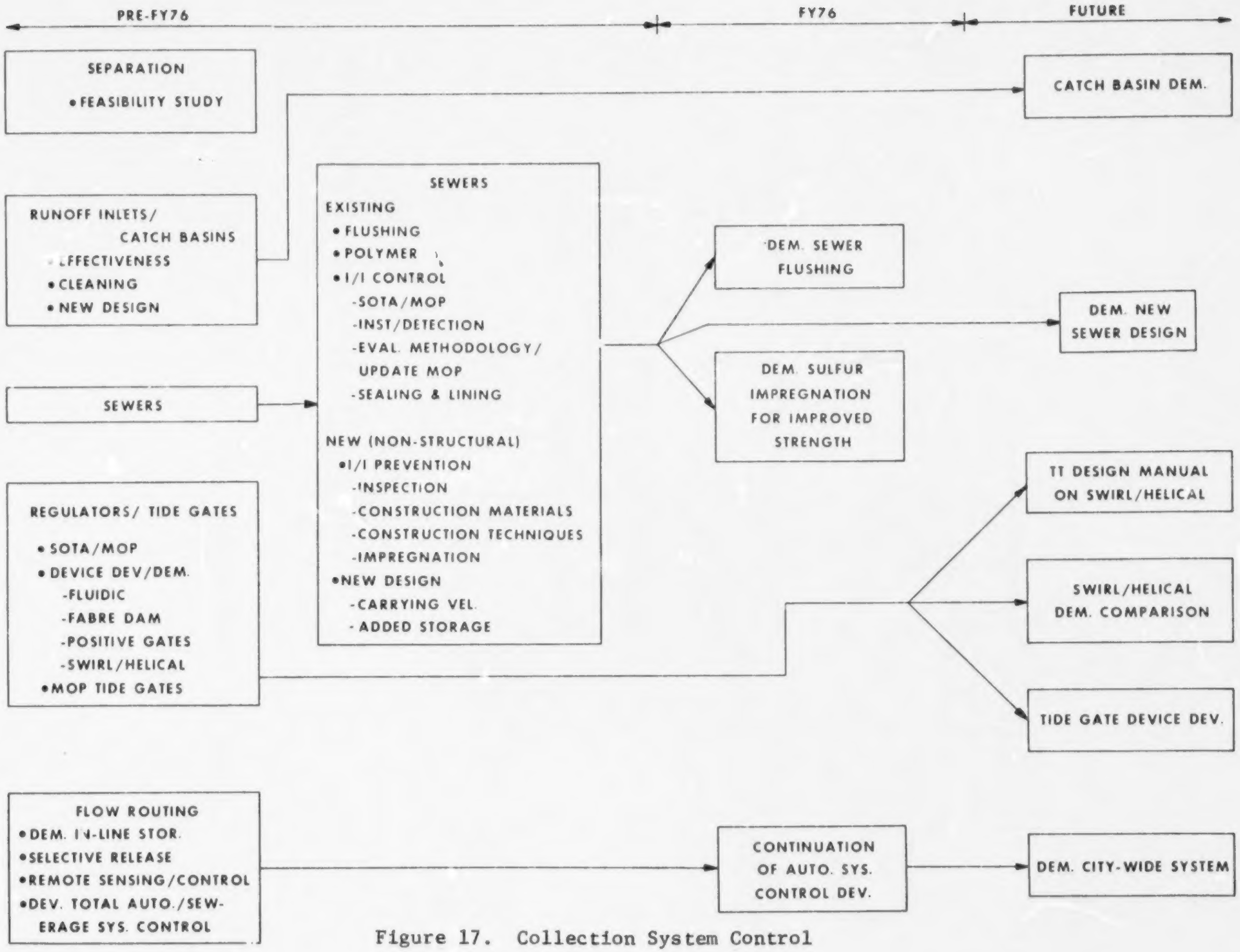


Figure 17. Collection System Control

as a short or long-term correction of troublesome pollution-causing conditions such as localized flooding and excessive overflows. Direct cost savings may be realized by eliminating relief sewer construction (6); however, additional cost verification at the site is necessary.

Infiltration/Inflow -- The Program SOTA (27) and manual of practice (MOP) (28) on infiltration/inflow (I/I) identified a significant problem which led to fruitful countermeasure research and a national emphasis on I/I control. Program developments have included detection methodology and instrumentation (27, 28, 10); preventive installation and construction techniques, new and improved materials (22, 27, 28, 52, 61, P-31, P-41); and correction techniques (12). A project to update and develop practices for determining and correcting infiltration and its economic analysis (P-18, 166) is nearing completion. An in-house paper on the analysis and evaluation of I/I has been published (R-14). Another project is evaluating the strength increases and erosion resistance, and resulting infiltration prevention from sulfur impregnation of concrete pipe (P-30, 52). Since pipe costs are significant, an increase in strength could lead to a decrease in pipe materials and construction costs.

Flow Routing

Another collection system control method is in-sewer or in-line storage and routing of storm flows to make maximum use of existing interceptors and sewer line capacity. The general approach comprises remote monitoring of rainfall, flow levels, and sometimes quality, at selected locations in the network, together with a centrally computerized console for positive regulation. This concept has proved to be effective in Minneapolis-St. Paul (19), Detroit (40, 118), and Seattle (29, 98). Seattle results are discussed later (Section 7, pp 56-58) to indicate potential control and cost benefits.

An ongoing project mentioned earlier with the City of San Francisco is developing an automatic operational model for real-time control (P-25). Future demonstration of the system is anticipated.

Regulators and Tide Gates

Two early publications in the area of flow regulator technology were the SOTA (23) and MOP (24).

Conventional regulators malfunction and cause excessive overflows. The new improved devices such as fluidic and positive control regulators have been developed and demonstrated (P-7, 9, 23, 24, 98, 173). The swirl and helical regulator devices are significant enough to single out separately.

Swirl and Helical Device Development -- The dual functioning swirl device has shown outstanding potential for providing both quality and quantity control (R-15, 93).

A swirl flow regulator/solids-liquid separator has been demonstrated in Syracuse, NY (P-2; R-16; 140a, Chapter II, pp 99-117). Figure 18 is an isometric view. The device, of simple annular shape construction, requires no moving parts. It provides a dual function, regulating flow by a central circular weir while simultaneously treating combined wastewater by a "swirl" action which imparts liquid-solids separation. The low-flow concentrate is diverted via a bottom orifice to the sanitary sewerage system for subsequent treatment at the municipal works, and the relatively clear supernatant overflows the weir into a central downshaft and receives further treatment or is discharged to the stream. The device is capable of functioning efficiently over a wide range (80:1) of combined sewer overflow rates, and can effectively separate suspended matter at a small fraction of the detention time required for conventional sedimentation or flotation (seconds to minutes as opposed to hours by conventional tanks). Tests indicate at least 50 percent removal of suspended solids and BOD. Tables 10 and 11 contain further treatability details on the Syracuse prototype. The captial cost of the 6.8 mgd Syracuse prototype was $55,000 or $8,100/mgd and $1,000/ac which makes the device highly cost-effective.

The swirl concept (for dual dry/wet weather flow treatment) has been piloted as a degritter (P-71) in Denver, CO and as a primary clarifier in Toronto, Canada (P-71). Test results are very encouraging and the concept has been further developed for erosion control.

A helical or spiral-type regulator/separator has also been developed based on principles similar to those of the swirl device. Its solids separation action is created by only a bend in the sewer line. It requires more land than the swirl device but may be preferred where available hydraulic head is limited.

Swirl and Helical: Products -- Important products for this category are design manuals for the swirl (69, 93, 101) and helical (132) regulator/separators, swirl degritter (99) and swirl erosion control devices (151); and a Technology Transfer Capsule Report (162) which ties the various swirl applications together.

Maintenance

Improved sewerage system inspection and maintenance is absolutely necessary for a total system approach to municipal water pollution control. We cannot afford the upgrading and proper operation of sewage treatment plants while a significant amount of sewage leaks into streams at the upstream points in the sewer network! Premature overflows and backwater intrusions during dry as well as wet weather caused by malfunctioning regulators and tide gates, improper diversion settings, and partially filled interceptors can thus be alleviated. Although the resulting abatement obtained is from a non-structural approach, it must be viewed as an ancillary benefit of required system maintenance. Regulator agencies should be anxious to strive for policy to enforce collection system maintenance.

STORAGE

Storage is perhaps the most cost-effective method available for reducing pollution resulting from combined sewer overflows and managing urban stormwater runoff. Furthermore, it is the best documented abatement measure in present practice. Program technological advancement for this category is depicted on Figure 19. (Storage with the resulting sedimentation that occurs, can also be thought of as a treatment process).

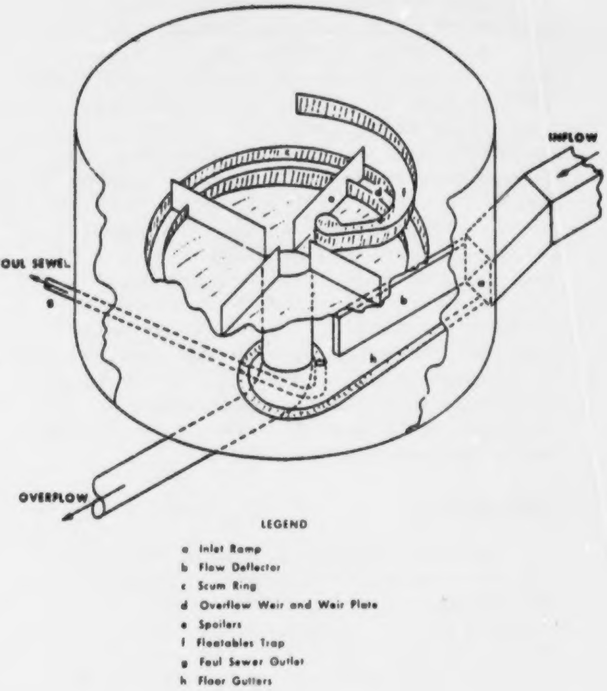


Figure 18 - Isometric View of Swirl Regulator/Concentrator

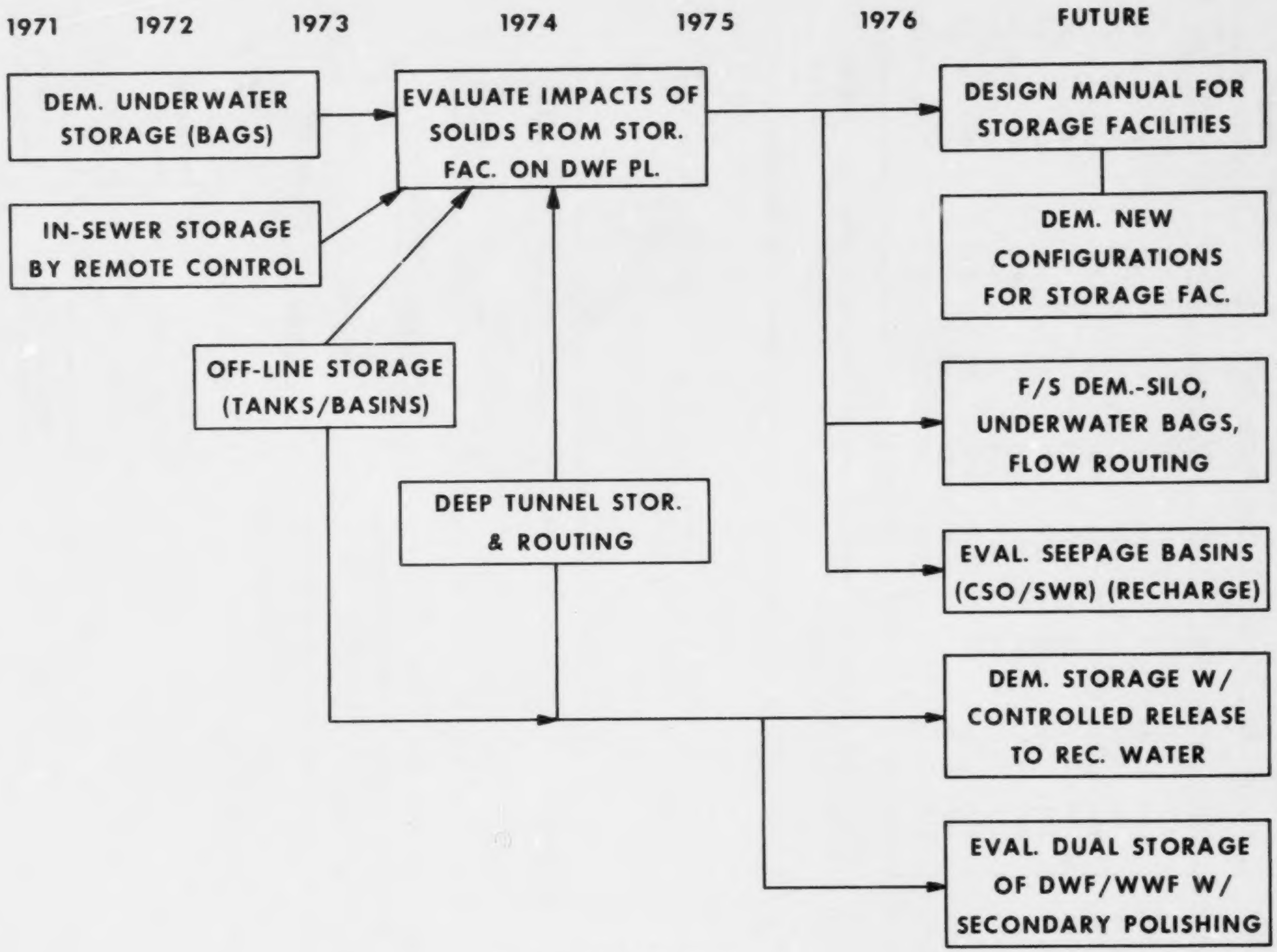

1971
1972
1973
1974
1975
1976
FUTURE
DEM. UNDERWATER STORAGE (BAGS)
IN-SEWER STORAGE BY REMOTE CONTROL
OFF-LINE STORAGE (TANKS/BASINS)
EVALUATE IMPACTS OF SOLIDS FROM STOR. FAC. ON DWF PL.
DEEP TUNNEL STOR. & ROUTING
DESIGN MANUAL FOR STORAGE FACILITIES
DEM. NEW CONFIGURATIONS FOR STORAGE FAC.
F/S DEM.-SILO, UNDERWATER BAGS, FLOW ROUTING
EVAL. SEEPAGE BASINS (CSO/SWR) (RECHARGE)
DEM. STORAGE W/ CONTROLLED RELEASE TO REC. WATER
EVAL. DUAL STORAGE OF DWF/WWF W/ SECONDARY POLISHING

Figure 19. Storage

TABLE 10 - SWIRL REGULATOR/CONCENTRATOR: SUSPENDED SOLIDS REMOVAL

	SWIRL CONCENTRATOR						CONVENTIONAL REGULATOR		
	Mass Loading kg			Average SS per storm, mg/l			Mass Loading kg		
Storm No.	Inf.	Eff.	% Rem.[b]	Inf.	Eff.	% Rem.[b]	Inf.	Underflow	% Rem.[a]
02-1974	374	179	52	535	345	36	374	101	27
03-1973	69	34	51	182	141	23	69	33	48
07-1974	93	61	34	110	90	18	93	20	22
10-1974	256	134	48	230	164	29	256	49	19
14-1974	99	57	42	159	123	23	99	26	26
01-1975	103	24	77	374	167	55	103	66	64
02-1975	463	167	64	342	202	41	463	170	34
06-1975	112	62	45	342	259	24	112	31	27
12-1975	250	168	33	291	232	20	250	48	19
14-1975	83	48	42	121	81	33	83	14	17
15-1975	117	21	82	115	55	52	117	72	61

[a] For the conventional regulator removal calculation, it is assumed that the SS concentration of the foul underflow equals the SS concentration of the inflow.

[b] Data reflecting negative SS removals at tail end of storms not included.

TABLE 11 - SWIRL REGULATOR/CONCENTRATOR: BOD_5 REMOVAL

	Mass Loading, kg			Average BOD_5 per storm, mg/l		
Storm No.	Influent	Effluent	% Rem.	Influent	Effluent	% Rem.
7-1974	277	48	82	314	65	79
1-1975	97	30	69	165	112	32
2-1975	175	86	51	99	70	29

The concept is to capture wet-weather flow and bleed it back to the treatment plant during low flow dry-weather periods. The result of controlling overflow by detention is shown on Figure 20. Notice how an entire hypothetical overflow event at point A is prevented by storage with controlled dewatering.

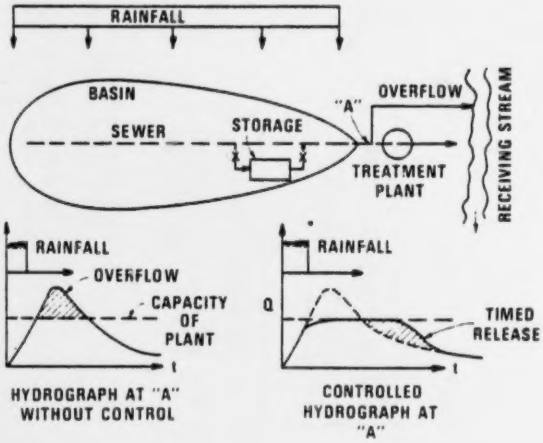


Figure 20 - Results of Controlling Storm Flow by Storage

Storage facilities possess many of the favorable attributes desired in combined sewer overflow control: (1) they are basically simple in structural design and operation; (2) they respond without difficulty to intermittent and random storm behavior; (3) they are relatively unaffected by flow and quality changes; and (4) they are capable of providing flow equalization and, in the case of sewers and tunnels, transmission. (Frequently they can be operated in concert with regional dry-weather flow treatment plants for benefits during both dry- and wet-weather conditions (107)). Finally, storage facilities are relatively fail-safe and adapt well to stage construction.

Storage facilities may be constructed in-line or off-line; they may be open or closed; they may be constructed inland and upstream, or on the shoreline; they may have auxiliary functions, such as flood protection, sewer relief, and flow transmission. (And they may be used for hazardous spill containment during dry weather.)

Disadvantages of storage facilities are their large size and dependency on other treatment facilities for dewatering and solids disposal.

Storage concepts investigated by the program include the conventional concrete holding tanks (18, 134) and earthen basins (30, 72); and the minimum land requirement concepts of: tunnels (40), underground (85) and underwater containers (15, 25, 26), underground "silos (96)," gravel packed beds with overhead land use (154), natural (85) and mined under and above ground formations, and the use of abandoned facilities and existing sewer lines (19, 29, 98, 118).

A 3.5 mg asphalt-lined storage basin in Chippewa Falls, WI (72) was constructed on reclaimed land and eliminated 59 out of 62 overflows during the evaluation periods.

Inherent in many of these storage schemes is the pumping/bleeding back of the stored flow to the DWF plant during off-peak hours. The impacts of this increased load on the DWF plant (both from a hydraulic and increased solids point of view) is an important consideration and has been investigated in an ongoing project (159, 161).

The feasibility of off-line storage and deep tunnel storage along waterways for selective discharge based on least receiving water impacts is presently being investigated in Rochester, NY (P-15). It is envisioned that this concept along with dual DWF/WWF storage, will be demonstrated in post FY 76 plans as part of a tie-in to construction grants.

Future Program plans include the investigation of new storage configurations, e.g., floating storage facilities, cofferdams, storage under piers, etc. Full-scale demonstration of some of the more promising configurations, such as silos and underwater bags, is also desirable.

Treatment

Due to adverse and intense flow conditions and unpredictable shock loading effects, it has been difficult to adapt existing treatment methods to storm-generated overflows, especially the microorganism dependent biological processes. The newer physical/chemical treatment techniques have shown more promise in overcoming these adversities. To reduce capital investments, projects have been directed towards high-rate operations approaching maximum loading boundaries. Applications include pretreatment or roughing, main or sole treatment, and particularly with microstrainers and filters, polishing devices.

The various treatment methods which have been developed and demonstrated by the Program for storm flow include physical and physical-chemical, biological, and disinfection (Figure 21). These processes, or combinations of these processes, can be adjuncts to the existing sanitary plant or serve as remote satellite facilities at the outfall.

Physical/Chemical Treatment

Physical processes with or without chemicals, such as: fine screens (34, 37, 38, 78, 105); swirl primary separators (162, P-29) and swirl degritters (99, 162, 158, P-29); high-rate filters (35, P-39); sedimentation (36, 81); and dissolved air flotation (20, 21, 131); have been developed and demonstrated by the Program.

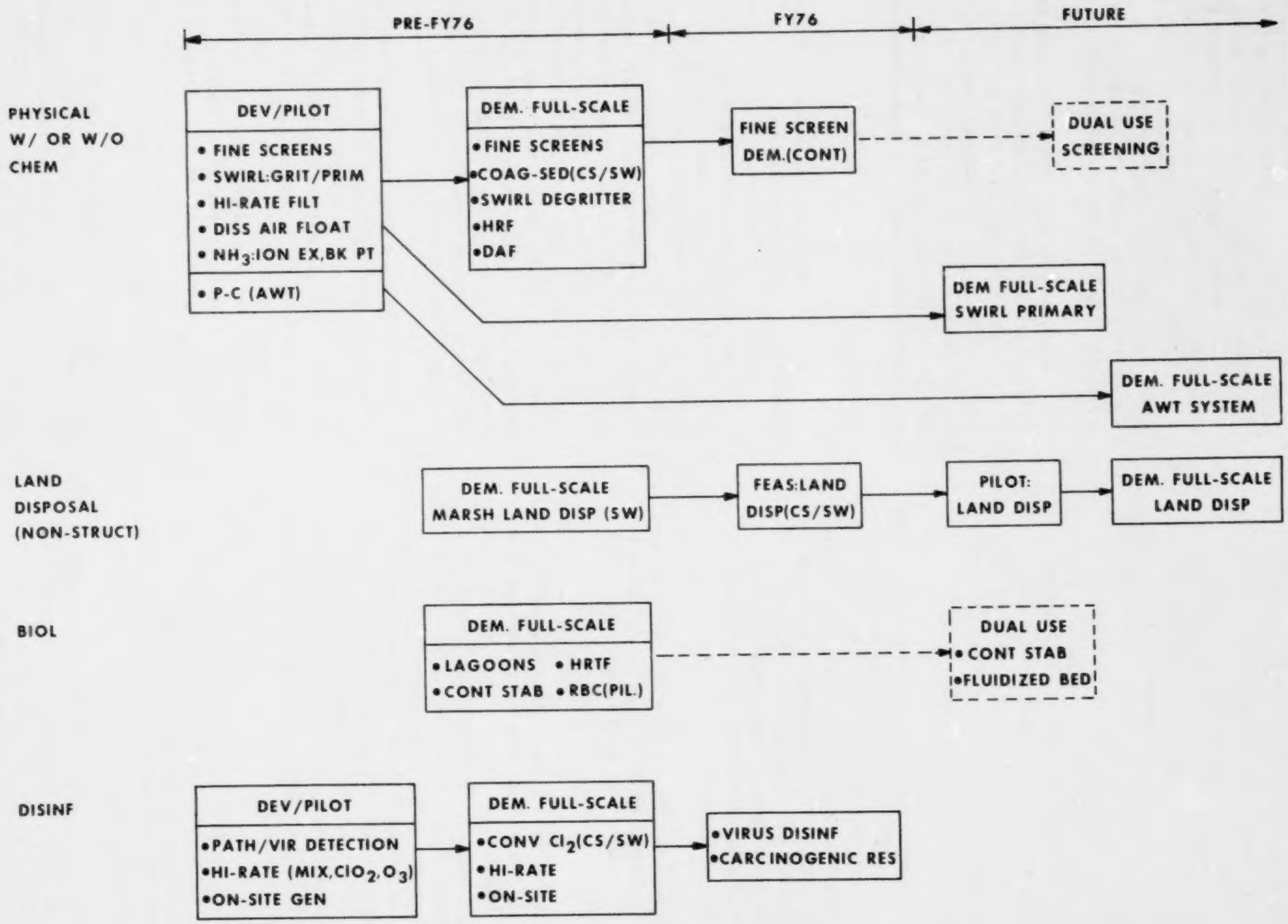


Figure 21. Treatment

Ammonia removal (P-12 and advanced physical-chemical-adsorption systems (81) have also been developed and tested at the pilot level Physical processes have shown importance for stormwater treatment because of their adaptability to automated operation, rapid startup and shut-down characteristics, high-rate operation, and very good resistance to shock loads.

A microstrainer is conventionally designed for polishing secondary sewage plant effluent at an optimum rate of approximately 10 gpm/sg. ft. Tests on a pilot microscreening unit of 23 micron aperture in Philadelphis have shown that at high influx rates of 25-30 gpm/ sg. ft., suspended solids removals in combined overflows as high as 90% can be achieved (34, 78, 105). Comparison of three different fine screens is continuing at Syracuse, NY (P-2).

A study in Cleveland (35) showed high potential for treating combined sewer overflows by in-pipe coagulation-filtration using anthrafilt and sand in a 7 foot deep bed. With the high loadings of 16 to 32 gpm/sg. ft. surface area, removal of solids was effectively accomplished throughout the entire depth of filter column. Test work showed suspended solids removal up to and exceeding 90 percent and BOD removals in the range of 60 to 80 percent. Substantial reductions, in the order of 30 to 80 percent of phosphates, can also be obtained. A large-scale high-rate filtration unit in New York City is being evaluated for the dual-treatment of dry and wet-weather flows (P-39).

Results from a 5.0 mgd screening and dissolved-air flotation demonstration pilot plant in Milwaukee (20), indicate that greater than 70 percent removals of BOD and suspended solids are possible. By adding chemical coagulants, 85 to 97 percent phosphate reduction can be achieved as an additional benefit. Based on these findings two full-scale prototypes (20 and 40 mgd) have been demonstrated in Racine, WI (P-23).

Land Disposal

As previously discussed, the use of marshlands for disposal of stormwater has been demonstrated in Minnesota. The feasibility of land disposal of raw CSO has also been investigated (161). Because of the cost of collection and transportation and large land requirements this concept does not appear feasible. Land disposal of CSO sludges, liquid or dewatered, appears feasible and promising for ultimate sludge disposal; however, further investigation in this area is required.

Biological Treatment

The following biological processes have been demonstrated: contact stabilization (117), high-rate trickling filtration (95), rotating biological contactors (106), and lagoons (108, 30). The processes have had positive evaluation, but with the exception of long term storage lagoons, must operate conjunctively with DWF plants to supply biomass, and require some form of flow equalization.

Disinfection

Because disinfectant and contact demands are great for storm flows, research has centered on high-rate applications by mixing and more rapid oxidants, i.e., chlorine dioxide (ClO_2) and ozone (O_3); and on-site generation (149, 31, 34, 78, 94, 105, 119). A continuing effort in Syracuse, NY (P-2) will involve viral disinfection and carcinogenic chlorine residual compound studies.

Treatment Process Performance

Treatment process performance in terms of design influx rate (gpm/sq ft) and BOD_5 and suspended solids (SS) removal efficiency is provided in Table 12. The high-rate performance of the swirl, microstrainer, filter, and dissolved air flotation is apparent when compared to sedimentation.

Sludge Solids

Due to the documented deleterious effect of CSO on the quality of receiving waters, WWF sludge handling and disposal has been given less emphasis previously in concession to the problems of treating the combined overflow itself. Sludge handling and disposal should be considered an integral part of CSO treatment because it significantly affects the efficiency and cost of the total waste treatment system.

TABLE 12 - WET-WEATHER TREATMENT PLANT PERFORMANCE DATA

Device	Control Alternatives	Design Loading Rate (gpm/ft^2)	Removal Efficiency (%)	
			BOD_5	SS
Primary	Swirl Concentrator[a,b]	60	25-60	50
	Microstrainer[c]	20	40-60	70
	High-Rate Filtration[d]	24	60-80	90
	Dissolved Air Flotation[d]	2.5	50-60	80
	Sedimentation[e]	0.5	25-40	55
	Representative Performance		40	60
Secondary	Contact Stabilization[f]		75-88	90
	Physical-Chemical[g]		85-95	95
	Representative Performance		85	95

[a] Field, 1976 (R-16)

[b] Sullivan, 1974 (101)

[c] Maher, 1974 (105)

[d] Lager and Smith, 1974 (102); w/chem. add.; hi-rate filter including pre-screens

[e] Performance data based on domestic wastewater treatment

[f] Agnew, et al., 1975 (117)

[g] Esimate based on performance of these units for domestic wastewater (102)

Flow characterization studies show that the annual quantity of CSO solids is at least equal to and in most cases greater than solids from DWF. For example, 29% of the sewered population in the U.S. is served by combined sewers. This represents a service area of $3x10^6$ acres. Assuming an average yearly rainfall of 36 in. and 50% of the runoff resulting in an overflow, the yearly volume of CSO in the U.S. would be $1.5x10^{12}$ gal. The corresponding average yearly volume of sludge resulting from treatment of all CSO nationwide is estimated at $41x10^9$ gal or 2.8% of the volume tested. The average solids content of this sludge would be about 1%. In comparison, an average yearly volume of dry weather sludge of $35x10^9$ gal may be expected from the same service area. Consequently, if nationwide CSO treatment was instituted there would be a problem equal to or greater than the problem there now is with municipal sludge.

The chronology of the Program's WWF sludge/solids technological advancement is contained in Figure 22. The need for defining the problem was recognized and, in FY 73, a contract was awarded (P-21) to characterize and preliminarily quantify CSO sludge/solids and perform treatability studies. Sludge handling/disposal techniques are also being evaluated and a nationwide assessment of the sludge problem has been conducted (P-24). As part of this assessment, the "impacts" of the following alternatives

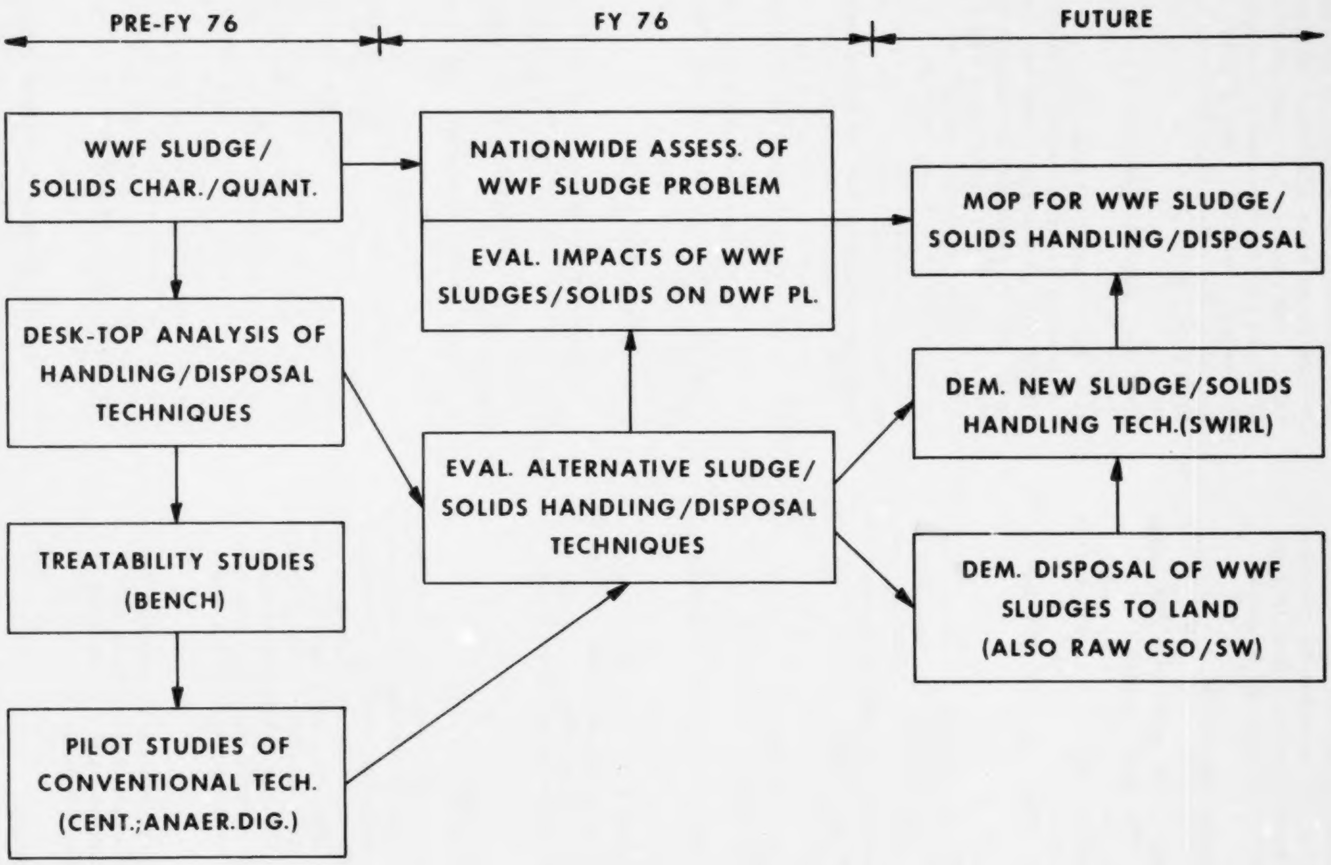

PRE-FY 76
FY 76
FUTURE
WWF SLUDGE/ SOLIDS CHAR./QUANT.
DESK-TOP ANALYSIS OF HANDLING/DISPOSAL TECHNIQUES
TREATABILITY STUDIES (BENCH)
PILOT STUDIES OF CONVENTIONAL TECH. (CENT.;ANAER.DIG.)
NATIONWIDE ASSESS. OF WWF SLUDGE PROBLEM
EVAL. IMPACTS OF WWF SLUDGES/SOLIDS ON DWF PL.
EVAL. ALTERNATIVE SLUDGE/ SOLIDS HANDLING/DISPOSAL TECHNIQUES
MOP FOR WWF SLUDGE/ SOLIDS HANDLING/DISPOSAL
DEM. NEW SLUDGE/SOLIDS HANDLING TECH.(SWIRL)
DEM. DISPOSAL OF WWF SLUDGES TO LAND (ALSO RAW CSO/SW)

Figure 22. Sludge/Solids

are being considered: bleed-back of the sludge to the municipal dry weather treatment plant, handling the sludges with parallel facilities at the dry weather plant, handling the sludges at the site of CSO treatment, and land disposal of either untreated or treated sludges.

Sludge: Products (Table 13)

Two reports are presently available (159, 161). The first covers the characterization, treatability, and quantification of CSO sludges and solids and the second is a "rough cut" at assessing the impact of handling and disposal.

TABLE 13 - SLUDGE/SOLIDS: PRODUCTS
- Characterization and Quantification of CSO Sludges and Solids (Draft report available) (159).
- WWF Sludge/Solids Impact Assessment (Preliminary report available) (161).
- WWF Sludge/Solids Treatability Studies (159).

The characterization, quantification, and treatability evaluation of sludges from separate stormwater will be done in the future.

Integrated Systems

By far the most promising and common approaches to urban stormwater management involve the integrated use of control and treatment with an areawide multidisciplinary perspective. When a single method is not likely to produce the best possible answer to a given pollution situation, various treatment and control measures may be combined for maximum flexibility and efficiency.

Integrated systems is divided into (1) Storage/Treatment, (2) Dual Use WWF/DWF Facilities, and (3) Control/Treatment/Reuse (Figure 23).

Storage/Treatment

Where there is storage, there is treatment by settling, pumpback to the municipal works, and sometimes disinfection; and treatment, which receives detention, provides storage. In any case, the breakeven economics of supplying storage must be evaluated when treatment is considered (35). The program has demonstrated all of these storage-treatment concepts on a full-scale basis (15, 18, 25, 26, 29, 30, 40, 72, 102, 108, 114, 118, 134, 146, 147, 154, P-10, P-37).

Dual Use, WWF/DWF Facilities

The concept of dual use is -- maximum utilization of wet-weather facilities during nonstorm periods and maximum utilization of dry-weather facilities during storm flows for total system effectiveness. The program has demonstrated the dual use of high-rate trickling filters (95), contact stabilization (117), and equalization basins (107, 114). On a pilot scale the Program has evaluated advanced physical-chemical treatment (81); and is in the process of demonstrating large-scale, high-rate filtration (P-39).

It should also be mentioned that combined sewers themselves are dual use systems.

Control/Treatment/Reuse

The sub-category, "Control/Treatment/Reuse" is a "catch-all" for all integrated systems. As the prime consideration, it is reasonable to apply the various nonstructural and land management techniques to reduce downstream loads and treatment costs.

Previous projects have evaluated the reuse of stormwater runoff for aesthetic, recreastional, and subpotable and potable water supply purposes (62, 79). In Mt. Clemens, MI, a series of three "lakelets" has been incorporated into a CSO treatment-park development (114). Treatment and disinfection is being provided so that these lakes are aesthetically pleasing and allow for recreation and reuse for irrigation. Other projects have shown the feasibility of reclaiming stormwater (3, 39).

The previously mentioned Houston project (P-16) is focusing on how a "natural drainage system" can be integrated into a resue scheme for recreation and aesthetics.

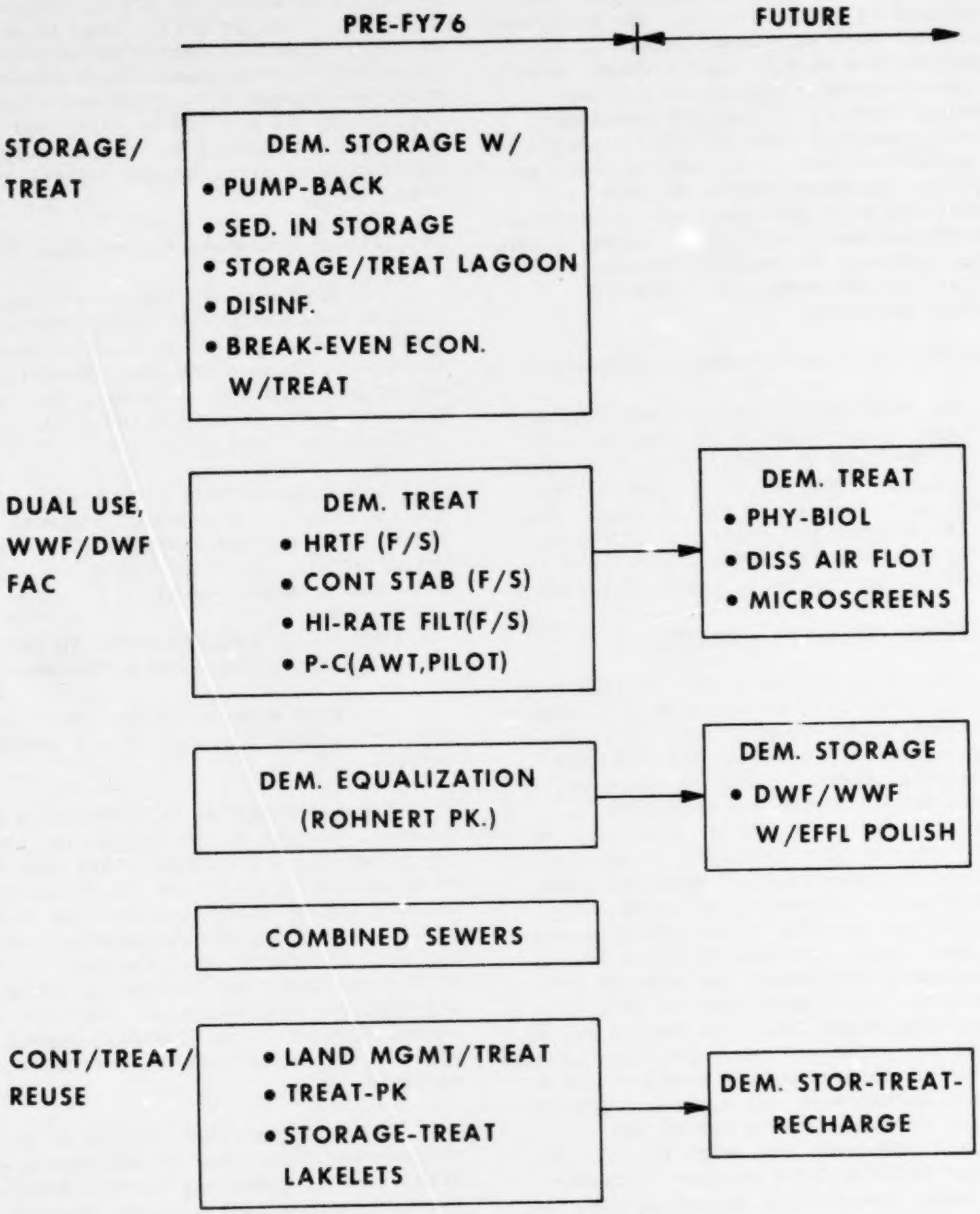


Figure 23. Integrated Systems

Integrated Systems: Products

The specific outputs from the integrated systems work have been predominantly in the form of demonstrations, documented by final reports. The previously referenced SOTA Assessment Report (102) summarizes the work in this area and ties it into wastewater management systems planning, design, and program implementation. Specific demonstration products are classified into main and complementary units for interrelating storm flow devices and unit processes and interfacing with dry-weather facilities. In the future it is important to evaluate storage used for DWF and WWF along with secondary effluent polishing.

TECHNICAL ASSISTANCE/TECHNOLOGY TRANSFER

The Technology Transfer area covers the formal dissemination of Program findings in the form of actual project reports, films, journal papers, SOTA reports, and manuals of practice and instruction. To date the Program has published approximately 160 reports, and it is the intent here to concentrate on the "user" type of document.

Significant Documents Completed

Reports generated by the program have received widespread recognition both within this country and abroad. Many have been referenced by EPA headquarters and used for 201/208 studies. Some of the more significant documents are indicated in Table 14. The first set of reports, item No. 1, set the pace for EPA's Program by identifying stormwater and combined sewer overflow as major sources of water pollution and provided a characterization data base (Refs. see item 1, Table 14). As previously mentioned, the manuals of practice on infiltration/inflow (27, 28, 97) and regulators (23, 24), Nos. 3 and 4, flagged two prime and basic problems leading to fruitful countermeasure research and a national emphasis on I/I control. Specific research products coming out of the regulator MOPs were the swirl (69, 93, 99, 101) and helical (132) devices -- resultant design manuals are listed as Nos. 5 and 6. Number 8 cites two instrumentation reports (130, 133) for flow analysis which have proven to be highly useful to the engineering community, including Construction Grants. An assessment of the significant impacts of highway deicing chemical use (67, 86) and practicable MOPs on control through proper salt storage and use (100, 104) are covered by items 9 and 10. Nos. 11 through 18 relate to Approach and Solution Methodology, the goal of the program. Item 19 refers to two very important user's manuals containing relatively simple urban runoff assessment planning methods (148, 153) which can be applied to 201 and 208 studies; and item 20 cites the previously mentioned national assessment of urban runoff control and costs (157).

Significant Documents Anticipated (Table 15)

In the immediate future a construction and O&M cost estimating manual (156) for CSO storage and treatment will be released, along with three other assessments: two on WWF sludge handling, disposal, and impact problems (159, 161), and the other on pathogens in stormwater (160).

Ongoing work will also lead to an updated SOTA and a planning document providing guidance and examples for total municipal studies (P-5) and a refined SWMM user's manual (P-53).

CAPITAL COSTS COMPARISON FOR STORM AND COMBINED SEWER CONTROL/TREATMENT

Table 16 shows a capital cost comparison for various SCS control and treatment methods.

Sewer separation is very costly with a national average of $20,000/ac (2, 102). In-system control storage costs were found to be as low as $0.02 and $0.25/gal for Detroit and Seattle, respectively (R-6d, 111). These figures represented 1/10th the cost for large off-line facilities, and 1/25th the costs for separation in Detroit and Seattle, respectively. Off-line storage varies from $0.03 to $4.75/gal depending on whether earthen or concrete basins are employed (102).

Per acre costs can only be given in wide ranges since they significantly vary with climate, receiving water, terrain, degree of urbanization, sewer network configuration, etc. Per capita and per acre unit costs may be applicable for gross estimating; but it is best to fix unit costs per gallon for storage and per mgd for treatment as design factors for the user engineer confronted with site-specific conditions.

TABLE 14 - SIGNIFICANT DOCUMENTS COMPLETED

1. Assessment - Problems of CSO/SW (2, 20, 34, 35, 41, 47, 51, 53, 54, 59, 60, 63, 65, 67, 73, 81, 82, 83, 88, 102, 112, 123, 124, 127, 128, 143, 149)
2. CSO/SW Seminar Reports (6, 40, 96, 140a)
3. MOP - I/I Prevention and Correction (27, 28, 97)
4. MOP - Regulation and Management (23, 24)
5. Design Manuals - Swirl: Regulator/Degritter/Erosion Control (69, 93, 99, 101)
6. Design Manual - Helical Regulator/Separator (132)
7. Assessments - Sources/Impacts of Urban Runoff Pollution (157, 164, 127, 88, 128, 73)
8. Assessments - Sampling/Flowrate Measurement (133, 130)
9. Assessment - Impact of Deicing (67, 86)
10. MOP's - Deicing Chemical Usage/Storage & Handling (100, 104)
11. Assessment - SOTA Urban Stormwater Management Technology (102, 111, 137)
12. User's Manuals - SWMM, Version I and II (44, 116)
13. Course Manual - SWMM Application (125)
14. SOTA - Urban Water Management Modeling (136)
15. MOP - Determination of Flowrates/Volumes (140)
16. Assessment/MOP - Stormwater Models (141)
17. MOP - Procedures for Stormwater Characterization/Treatment Studies (145)
18. MOP's - Sediment & Erosion Control (68, 70, 168, 169)
19. User's Manuals - Simplified Urban Runoff Planning Models/Tools (148, 153)
20. Assessment - Nationwide Stormwater/Characterization/Impacts/Costs (157)

TABLE 15 - SIGNIFICANT DOCUMENTS ANTICIPATED

- Estimating Manual - CSO Storage and Treatment Costs (156)
- User's Manual - SWMM Version III
- Assessments - WWF Sludge Handling/Disposal Problems/Impacts (159/161)
- Assessment - Pathogens in Stormwater and Combined Sewer Overflow (160)
- SOTA/Planning Guide - Update Storm and Combined Sewer Overflow Management and Treatment/Total Approach Methodology
- Design Manual - Swirl: Primary Treatment
- MOP - I/I Analysis, Prevention and Control
- Instruction Manual - Storm and Combined Sewer Overflow Technology

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- MOP - Pollution Control from Construction Activities
- MOP - Refined Solution Methodology
- MOP - Land Management
- Design Manual - Storage Facilities
- Consolidated Design Text - Swirl and Helical

Table 16. Typical Capital Costs for SCS Control/Treatment (ENR 2000)

COMPONENT DEVICES	$/GAL	$/MGD	$/ACRE
SEPARATION			10,000 (SEATTLE) 6,500 (DES MOINES) 32,000 (CLEVELAND) 20,000-NATIONAL AVE.
STORAGE			
• IN-LINE	0.02 - 0.25 (DETROIT) (SEATTLE)		400 (SEATTLE) 250 (MINNEAPOLIS)
• OFF-LINE			
-EARTHEN	0.03-0.26		
-CONCRETE TANKS	1.00-4.75		7,000 (JAMAICA, NYC)
TREATMENT			
• PHYSICAL W&W/O CHEMICALS			
-FINE SCREENING/MICROSTRAINING		5,000/12,000	2,000/13,000
-SEDIMENTATION		10,000-50,000	3,500-6,500
-HI-RATE FILT		70,000	10,000
-DISS AIR FLOAT		40,000	6,500 (MILWAUKEE)
-SWIRL		8,000 (SYRACUSE) 2,000 (LANCASTER)	500(SYRACUSE) 500(LANCASTER)
•BIOLOGICAL			
-CON. STAB/TRICK. FILTER		80,000	1,700
-LAGOONS		17,000	5,000
•PHYSICAL-CHEMICAL SYSTEMS		150,000-2×10^6	
•DISINFECTION			
-CONVENTIONAL		1,500	
-HI-RATE(STATIC MIXING)		900	
INTEGRATED SYSTEMS			
•STORAGE/TRMT/REUSE		1×10^6(KINGMAN LAKE)	10,000(KINGMAN LAKE)
-TREATMT-PARK CONCEPT		17,000(MT. CLEMENS)	5,000(MT. CLEMENS)
LAND MANAGEMENT			
•STRUCTURAL			
-DIVERSION BERMS			160
•NON-STRUCTURAL			
-STREET CLEANING			0.7

These data are based on a limited number of specific projects thus they represent only a range of placement. In extrapolating these costs into master plan systems for cities, the totals frequently approach $500 to $1,000 per capita.

Physical treatment costs range between $2,000 and $35,000/mgd; whereas physical with chemical treatment varies between $35,000 and $80,000/mgd. Biological treatment ranges between $17,000 and $80,000/mgd depending on whether land is available for lagooning or if contact stabilization or trickling filtration (102) is resorted to. As can be seen from the table, costs for the swirl at $2,000/mgd and $500/acre (P-4) are considerably lower than other forms of treatment installation.

Preliminary figures for incorporating land management techniques show a definite cost-effectiveness benefit.

It must be mentioned that the various alternatives offer different degrees of removal which will have a significant bearing on the selection process.

SEATTLE: IN-LINE STORAGE IS COST-EFFECTIVE

A case study illustrating cost-effectiveness by Seattle's flow routing approach is worthwhile discussing (98, 140a).

Costs

The Seattle in-line storage system covering 13,250 acres costs $5.3 million or $400 per acre as opposed to tens of thousands of dollars per acre for other alternatives. A specific Seattle study revealed $10,000 per acre for separation. The low cost is attributed to a quasi-structural system which takes advantage of the existing combined sewer network; and control gates installed at strategic points only. The system is highly signal and computer oriented with minimal hardware requirements. In fact, one-half the costs were for computers and related software. Of course, in-line storage is site specific since implementation of the concept requires a relatively large and flat existing combined system.

Pollutant Reduction

Overflow and pollutant reduction from 12 major overflow points averaged 55% and 68%, respectively. Also, 90% of the overflow volume was reduced by experimental automatic control.

Effectiveness

Effectiveness of the system is proven by a one to two mg/l D.O. increase and a 50% coliform reduction in the receiving water.

DES MOINES: CONTROL COSTS VERSUS D.O. VIOLATIONS

Based on a study for the City of Des Moines (157) using a simplified receiving water model, four control alternatives were compared considering cost and true effectiveness in terms of frequency of D.O. standard violations.

As Table 17 depicts, 25% BOD removal of WWF coupled with secondary treatment of DWF results in slightly higher D.O. levels in the receiving water than tertiary treatment and no control of urban runoff. The annual cost of 25% BOD removal for WWF is 10% of the cost for tertiary treatment. However, existing DWF treatment facilities exhibit a comparable effect to these two options at no additional cost. However, significant increase in the minimum D.O. levels of the Des Moines River is obtained by 75% BOD removal of WWF with the annual cost of this level of control being lower than the cost of tertiary treatment. The application to Des Moines demonstrated clearly the overwhelming effect of urban runoff pollution on critical D.O. concentrations. The cost-effectivenss of various treatment alternatives can be determined realistically only by a continuous analysis of the frequency of water quality violations.

In the selection of the "best" control strategy, other factors that may become important are: (1) recovery of receiving waters from shock loads caused by runoff, (2) local and regional water quality goals, and (3) public willingness to pay the costs associated with each level of control.

TABLE 17 - DES MOINES: CONTROL COSTS VERSUS VIOLATIONS OF DO STANDARD (4 PPM)

Options	Total Annual Cost ($/yr)	% of Precipitation Events Violating Standard
1. DWF Tertiary Treatment	6.3M	40
2. WWF 25% BOD Removal	0.6M	30
3. WWF 75% BOD Removal	4.0M	3
4. DWF Secondary Trt Only	0	42

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*Copies may be obtained from EPA Storm & Combined Sewer Section, Edison, NJ 08817

Note: Number in left margin corresponds to reference numbers cited in report text.

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88a	EPA-600/2-73-002	A Portable Device for Measuring Wastewater Flow in Sewers: by Michael A. Nawrocki, Hittman Associates, Inc., Columbia, MD	GPO EP 1.23/2:600/2-73-002 NTIS PB 235 634

*Copies may be obtained from EPA Storm and Combined Sewer Section, Edison, NJ 08817

Ref. No.	Report Number	Title/Author	Source
89.	EPA-660/2-73-035	Joint Construction Sediment Control Project: by B.C.Becker, et al., Water Resources Administration, State of Maryland	GPO EP 1.23/2:660/2-73-035 NTIS PB 235 634
90.	EPA-660/2-74-071	Programmed Demonstration for Erosion and Sediment Control Specialist: by T.R.Mills, et al., Water Resources Administration, State of Maryland	GPO EP 1.23/2:660/2-74-071
91.	EPA-660/2-74-072	Demonstration of the Separation and Disposal of Concentrated Sediments: by M.A.Nawrocki, Hittman Associates, Columbia, MD	GPO EP 1.23/2:660/2-74-072
92.	EPA-660/2-74-073	An Executive Summary of Three EPA Demonstration Programs in Erosion and Sediment Control: by B.C.Becker et al., Hittman Associates, Columbia, MD	GPO EP 1.23/2:660/2-74-073
93.	EPA-670/2-73-059	*The Dual-Functioning Swirl Combined Sewer Overflow Regulator/Concentrator: by R.Field, USEPA, Edison, NJ	GPO EP 1.23/2:670/2-73-059 NTIS PB 227 182/3
94.	EPA-670/2-73-067	*Hypochlorination of Polluted Stormwater Pumpage at New Orleans: by U.R. Pontius, E.H.Pavis, Byrne Engineering Corp., New Orleans, LA	GPO EP 1.23/2:670/2-73-067 NTIS PB 228 581
95.	EPA-670/2-73-071	*Utilization of Trickling Filters for Dual-Treatment of Dry and Wet-Weather Flows: by P.Homack, et al., E.T. Killam Assoc., Inc., Millburn, NJ	GPO EP 1.23/2:670/2-73-071 NTIS PB 231 251
96.	EPA-670/2-73-077	*Combined Sewer Overflow Seminar Papers: by Storm and Combined Sewer Technology Branch, USEPA, Edison, NJ	GPO EP 1.23/2:670/2-73-077 NTIS PB 231 836
97.	EPA-670/9-74-004	*Excerpts from "Control of Infiltration and Inflow into Sewer Systems," and "Prevention and Correction of Excessive Infiltration and Inflow into Sewer Systems Manual of Practice, January 1971." Complete reports can be purchased from NTIS, See PB numbers listed on third page of this Bibliography.	NTIS Pending
98.	EPA-670/2-74-022	*Computer Management of a Combined Sewer System: by C.P.Leiser, Municipality of Metropolitan Seattle, Seattle, WA	GPO EP 1.23/2:670/2-74-022 NTIS PB 235 717

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Ref. No.	Report Number	Title/Author	Source
99.	EPA-670/2-74-026	*The Swirl Concentrator as a Grit Separator Device: by R.H.Sullivan, et al., American Public Works Assoc., Chicago, IL	NTIS PB 233 964
100.	EPA-670/2-74-033	*Manual for Deicing Chemical Storage and Handling: by D.L.Richardson, et al., Arthur D. Little, Inc., Cambridge, MD	NTIS PB 236 152
101.	EPA-670/2-74-039	*Relationship between Diameter and Height for Design of a Swirl Concentrator as a Combined Sewer Overflow Regulator: by R.H.Sullivan, et al., American Public Works Assoc., Chicago, IL	NTIS PB 234 646
102.	EPA-670/2-74-040	*Urban Stormwater Management and Technology An Assessment: by J.A.Lager and W.G.Smith, Metcalf & Eddy, Inc., Palo Alto, CA	NTIS PB 240 687
103.	EPA-660/2-74-043	Prediction of Subsoil Erodibility Using Chemical, Mineralogical and Physical Parameters: by C.B.Roth, D.W.Nelson, M.J.M.Romkens, Cincinnati, OH	GPO EP 1.23/2:660/2-74-043 NTIS PB 231 846
104.	EPA-670/2-74-045	*Manual for Deicing Chemicals: Application Practices: D.L.Richardson, Arthur D. Little, Inc., Cambridge, MD	NTIS PB 239 694
105.	EPA-670/2-74-049	*Microstraining and Disinfection of Combined Sewer Overflows-Phase III: by M.B.Maher, Crane Co., King of Prussia, PA	GPO EP 1.23/2:670/2-74-049 NTIS PB 235 771
106.	EPA-670/2-74-050	Combined Sewer Overflow Treatment by the Rotating Biological Contactor Process: by F.L.Welsh, D.J.Stucky, Autotrol Corp., Milwaukee, WI	NTIS ONLY PB 231 892
107.	EPA-670/2-74-075	*Surge Facility for Wet- and Dry-Weather Flow Control: by H.L.Welborn City of Rohnert Park, CA	GPO EP 1.23/2:670/2-74-075 NTIS PB 238 905
108.	EPA-670/2-74-079	An Evaluation of Three Combined Sewer Overflow Treatment Alternatives: by J.W.Parks, et al., City of Shelbyville, IL	NTIS ONLY PB 239 115
109.	EPA-670/2-74-086	Chemical Impact of Snow Dumping Practices: by J.P.O'Brien, et al., Arthur D. Little, Inc., Cambridge, MD	NTIS ONLY PB 238 764

*Copies may be obtained from EPA Storm and Combined Sewer Section, Edison, NJ 08817

Ref. No.	Report Number	Title/Author	Source
110.	EPA-670/2-74-087	Assessment and Development Plans for Monitoring of Organics in Storm Flows: by A.Molvar, A.Tulmello, Raytheon Co., Portsmouth, RI	NTIS ONLY PB 238 810
111.	EPA-670/2-74-090	*Countermeasures for Pollution from Overflows: by R.Field, USEPA, and J.A.Lager, Metcalf & Eddy, Inc., Palo Alto, CA	NTIS PB 240 498
112.	EPA-670/2-74-096	*Characterization and Treatment of Urban Land Runoff: by Newton V. Colston, Jr., North Carolina State University, Raleigh, NC	NTIS PB 240 978
113.	EPA-670/2-75-002	*Suspended Solids Monitor: by John W. Liskowitz, et al., American Standard Inc., New Brunswick, NJ	NTIS PB 241 581
114.	EPA-670/2-75-010	*Multi-Purpose Combined Sewer Overflow Treatment Facility, Mt. Clemens, MI: by V.U.Mahida, F.J.DeDecker, Spalding DeDecker & Assoc., Madison Heights, MI	NTIS PB 242 914
115.	EPA-670/2-75-011	*Physical and Settling Characteristics of Particulates in Storm and Sanitary Wastewater: by R.J.Dalrymple, et al, Beak Consultants for American Public Works Assoc., Chicago, IL	NTIS PB 242 001
116.	EPA-670/2-75-017	*Stormwater Management Model User's Manual-Version II: W.C.Huber, et al., University of Florida, Gainesville, FL	NO NTIS
117.	EPA-670/2-75-019	*Biological Treatment of Combined Sewer Overflow at Kenosha, WI: by R.W.Agnew,et al., Envirex, Milwaukee, WI	NTIS PB 242 126
118.	EPA-670/2-75-020	*Sewage System Monitoring and Remote Control: by T.R.Watt, et al., Detroit Metro Water Department, Detroit MI	NTIS PB 242 107
119.	EPA-670/2-75-021	*Bench-Scale High-Rate Disinfection of Combined Sewer Overflows: by P.E. Moffa, et al., O'Brien & Gere Engrs., Syracuse, NY	NTIS PB 242 296
120.	EPA-670/2-75-022	*Urban Stormwater Management Modeling and Decision-Making: by J.P.Heaney and W.C.Huber, University of Florida, Gainesville, FL	NTIS PB 242 290

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Ref. No.	Report Number	Title/Author	Source
121.	EPA-670/2-75-035	Stream Pollution Abatement by Supplemental Pumping: by C.W.Reh and W.E. Saddler, City of Richmond, VA	NTIS ONLY PB 239 566
122.	EPA-670/2-75-041	*Storm Water Management Model: Dissemination and User Assistance: J.A.Hagerman and F.R.S.Dressler, University City Science Center (UCSC), Philadelphia, PA	NTIS PB 2242 544
123.	EPA-670/2-75-046	*Rainfall-Runoff Relations on Urban and Rural Areas: by E.F.Brater and J.D.Sherrill, University of Michigan, Ann Arbor, MI	NTIS PB 242 830
124.	EPA-670/2-75-054	Characterization and Treatment of Combined Sewer Overflows: by Engineering Science Inc. for City and County of San Francisco, CA	NTIS ONLY PB 241 299
125.	EPA-670/2-75-065	*Short Course Proceedings, Application of Stormwater Management Models: F.DiGiano, et al., University of Massachusetts, Amherst, MA	NTIS PB 247 163
126.	EPA-670/2-75-067	*Automatic Organic Monitoring System for Storm and Combined Sewers: by A.Tulumello, Raytheon Co., Portsmouth, RI	NTIS PB 244 142
127.	EPA-440/9-75-004	*Water Quality Management Planning for Urban Runoff: by G.Amy, et al., Woodward-Clyde, San Francisco, CA	NTIS PB 241 689
128.	EPA-600/2-75-004	*Contributions of Urban Roadway Usage to Water Pollution: by D.G.Shaheen, Biospherics Inc., Rockville, MD	NTIS PB 245 854
129.	EPA-600/2-75-007	Impact of Hydrologic Modifications on Water Quality: by J.Bhutani, et al., Mitre Inc., McLean, VA	NTIS PB 248 523
130.	EPA-600/2-75-027	*Sewer Flow Measurement-A State-of-the-Art Assessment: by P.E.Shelley and G.A.Kirkpatrick, EG&G Washington Analytical Services Center, Inc., Rockville, MD	NTIS PB 250 371
131.	EPA-600/2-75-033	*A Treatment of Combined Sewer Overflows by Dissolved Air Flotations: by T.A.Bursztynsky, et al., Engineering Science Inc., Berkeley, CA	NTIS PB 248 186

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Ref. No.	Report Number	Title/Author	Source
132.	EPA-600/2-75-062	*The Helical Bend Combined Sewer Overflow Regulator: by R.H.Sullivan, et al., American Public Works Assoc., Chicago, IL	NTIS PB 250 619
133.	EPA-600/2-75-065	*An Assessment of Automatic Sewer Flow Samplers-1975: by P.E.Shelley, EG&G Washington Analytical Services Center, Inc., Rockville, MD	NTIS PB 250 987
134.	EPA-600/2-75-071	*Detention Tank for Combined Sewer Overflow: by Consoer, Townsend and Associates, Milwaukee, WI	NTIS PB 250 427
135.	EPA-600/2-76-006	*Design and Testing of a Prototype Automatic Sewer Sampling System: by P.Shelley, EG&G Washington Analytical Service Center Inc., Rockville, MD	NTIS PB 252 613
136.	EPA-600/2-76-058	Future Direction of Urban Water Models: by M.Sonnen, Water Resources Engineers (WRE), Walnut Creek, CA	NTIS ONLY PB 249 049
137.	EPA-600/2-76-095	*Urban Runoff Pollution Control Program Overview: FY 76: R.Field, A.N.Tafuri, H.E.Masters, USEPA, Edison, NJ	NTIS PB 252 223 In-house Report
138.	EPA-600/2-76-105	*An Economic Analysis of the Environmental Impact of Highway Deicing: by D.M.Murray and U.F.W.Ernst, Abt Associates, Inc., Cambridge, MA	NTIS PB 253 268
139.	EPA-600/2-76-115	*A Passive Flow Measurement System for Storm and Combined Sewer: by K.Foreman, Grumman Ecosystems Corp., Bethpage, NY	NTIS PB 253 383
140.	EPA-600/2-76-116	*Urban Stormwater Runoff Determination of Volumes and Flowrates: by B.C.Yen and V.T.Chow, University of Illinois, Urbana, IL	NTIS PB 253 410
140a.	WPD 03-76-04	*Proceedings Urban Stormwater Management Seminars: Atlanta, GA, Nov. 4-6, and Denver, CO, Dec. 2-4, 1975, Edited by Dennis Athayde, USEPA, Water Planning Div., Washington, DC	NTIS PB 260 889
141.	EPA-600/2-76-175a	*Assessment of Mathematical Models for Storm and Combined Sewer Management: by A.Brandstetter, Battelle, Pacific Northwest Lab., Richland, WA	NTIS PB 259 597

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Ref. No.	Report Number	Title/Author	Source
142.	EPA-600/2-76-175b	Assessment of Mathematical Models for Storm and Combined Sewer Management-Appendix: by A.Brandstetter, Battelle, Pacific Northwest Lab., Richland, WA	NTIS ONLY PB 258 644
143.	EPA-600/2-76-217a	Urban Runoff Characteristics-Vol. I, Analytical Studies: by H.C.Preul and C.N.Papadakis, University of Cincinnati, Cincinnati, OH	NTIS ONLY PB 258 033
144.	EPA-600/2-76-217b	Urban Runoff Characteristics-Vol. II, by H.C.Preul and C.N.Papadakis, University of Cincinnati, Cincinnati, OH	NTIS ONLY PB 258 034
145.	EPA-600/2-76-145	*Methodology for the Study of Urban Storm Generated Pollution and Control: by Envirex, Environmental Sciences Division, Milwaukee, WI	NTIS PB 258 743
146.	EPA-600/2-76-222a	*Wastewater Management Program, Jamaica Bay-Vol.I, Summary Report: by D.L.Feurstein and W.O.Maddaus, City of New York, NY	NTIS PB 260 887
147.	EPA-600/2-76-222b	Wastewater Management Program, Jamaica Bay-Vol. II, Supplemental Data, NYC Spring Creek: by D.L. Feurerstein and W.O.Maddaus, City of New York, NY	NTIS ONLY PB 258 308
148.	EPA-600/2-76-218	*Development and Application of a Simplified Stormwater Management Model: by Metcalf & Eddy, Inc. Palo Alto, CA	NTIS PB 258 074
149.	EPA-600/2-76-244	*Proceedings of Workshop on Microorganism in Urban Stormwater: March 24, 1975, Storm and Combined Sewer Section, USEPA, Edison, NJ	NTIS Pending
150.	EPA-600/2-76-243	*Wastewater Flow Measurement in Sewers Using Ultrasound, Milwaukee: by R.J.Anderson and S.S.Bell, City of Milwaukee, WI	NTIS Pending
151.	EPA-600/2-76-271	*The Swirl Concentrator for Erosion Runoff Treatment: by R.H.Sullivan, et al., American Public Works Assoc., Chicago, IL	NTIS Pending
152.	EPA-600/2-76-242	*Development of a Hydrophobic Substance to Mitigate Pavement Ice Adhesion: by B.H.Alborn and H.C.Poehlmann, Ball Bros., Inc., Boulder, CO	NTIS Pending

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153.	EPA-600/2-76-275	*Storm Water Management Model Level I Preliminary Screening Procedures: by J.P.Heaney, et al., University of Florida, Gainesville, FL	NTIS PB 259 916
154.	EPA-600/2-76-272	*Demonstration of Void Space Storage with Treatment and Flow Regulation: by Karl R. Rohrer Assocs., Inc., Akron, OH	NTIS Pending
155.	EPA-600/2-76-228	Demonstration of Interim Techniques for Reclamation of Polluted Beachwater: by J.F.Weber, City of Cleveland, OH	NTIS ONLY PB 258 192
156.	EPA-600/2-76-286	*Cost Estimating Manual--Combined Sewer Overflow Storage Treatment: by H.H.Benjes, Jr., Culp, Wesner, Culp, Inc., El Dorado Hills, CA	NTIS Pending
157.		Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Vol. II: Cost Assessment and Impacts: by J.F.Heaney, et al., University of Florida, Gainesville, FL	At Printers
158.		Field Prototype Demonstration of the Swirl Degritter: by R.H.Sullivan, et al., American Public Works Assoc., Chicago, IL	DRAFT
159.		*Handling and Disposal of Sludges from Combined Sewer Overflow Treatment-Phase I (Characterization): by M.K. Gupta, et al., Envirex, Environmental Science Division, Milwaukee, WI	At Printers
160.		Microorganisms in Urban Stormwater: by V.P.Olivieri, et al., The Johns Hopkins University, Baltimore, MD	DRAFT
161.		Assessment of the Impact of the Handling and Disposal of Sludges Arising from Combined Sewer Overflow Treatment: by M.J.Clark and A.Geinopolos, Envirex, Environmental Sciences Division, Milwaukee, WI	DRAFT
162.		Swirl Device for Regulating and Treating Combined Sewer Overflows, EPA Technology Transfer Capsule Report: by R.Field and H.E.Masters, USEPA, Storm and Combined Sewer Sec., Edison, NJ	At Printers

*Copies may be obtained from EPA Storm and Combined Sewer Section, Edison, NJ 08817

Ref. No.	Report Number	Title/Author	Source
163.	EPA-600/2-77-033	Methods for Separation of Sediment from Storm Water at Construction Sites: by J.F.Ripken, et al., Univ. of Minnesota, Minneapolis, MN	NTIS ONLY Pending
164.		Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Vol. III: Characterization: by R.H.Sullivan, et al., American Public Works Assoc., Chicago, IL	DRAFT
165.		Cost-Effective Pollution Control of Combined Wastes and Urban Runoff: by Clinton Bogert Assocs., Fort Lee, NJ	DRAFT
166.		Analysis of Practices for Preparing an Economic Analysis and Determining Infiltration and Inflow: Vol. II: Manual of Practice, Sewer System Evaluation Rehabilitation and New Construction: by R.H.Sullivan, et al., American Public Works Assoc., Chicago, IL	DRAFT
167.	EPA-440/9-75-001	Report on State Sediment Control Institutes Program: USEPA, Office of Water Planning and Standards	NTIS PB 241 088
168.	EPA-600/8-76-001a	Erosion and Sediment Control Audio-Visual Training Program: Instruction Manual: by The State of Maryland Water Resources Administration; Dept. of Transportation, The Federal Highway Administration; The U.S. Department of Agriculture, Soil Conservation Service; and USEPA, Office of Research and Development	NTIS PB 256 901
169.	EPA-600/8-76-001b	Erosion and Sediment Control Audio-visual Training Program: Workbook: by The State of Maryland Water Resources Administration; Department of Transportation, The Federal Highway Administration; The U.S. Department of Agriculture, Soil Conservation Service; and USEPA, Office of Research and Development	NTIS PB 258 471
170.	EPA-600/2-77-015	*Treatment of Combined Sewer Overflows by High Gradient Magnetic Separation: by John Oberteuffer, et al., Sala Magnetics, Cambridge, MA	NTIS Pending

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171.		Cottage Farm Combined Sewer Detention and Chlorination Station, Cambridge, MA: by Commonwealth of MA Metropolitan District Commission, Boston, MA	NTIS ONLY Pending
172.		Bachman Treatment Facility for Excessive Storm Flow in Sanitary Sewers: by H.W.Wolf, Texas A&M University, for Dallas Water Utilities, Dallas, TX	NTIS ONLY Pending
173.		Evaluation of Fluidic Combined Sewer Regulators Under Municipal Service Conditions: by P.A.Freeman, Peter A. Freeman Assoc., Inc., Berlin, MD	At Printers
174.		Catchbasin Technology Overview and Assessment: by J.J.Lager, et al., Metcalf & Eddy, Inc., Palo Alto, CA, in association with Hydro-Research-Science, Santa Clara, CA	At Printers

ONGOING URBAN RUNOFF POLLUTION CONTROL PROJECTS

Project Reference Number	On-Going Projects
P-1	"Nationwide Characterization, Impacts, and Critical Evaluation of Combined Sewer Overflow, Stormwater, and Non-Sewered Urban Runoff." American Public Works Association, 68-03-0283
P-2	"Disinfection/Treatment of Combined Sewer Overflows - Syracuse, N.Y." Onondaga County, N.Y., 802400
P-3	"Development of a Swirl Concentrator and a Helical Combined Sewer Overflow Dual Functioning Regulator-Separator." American Public Works Association, 68-03-0272
P-4	"Demonstration of a Swirl Regulator/Solids Separator System for Control of Combined Sewer Overflows." City of Lancaster, Pennsylvania, 802219
P-5	"State-of-the-Art Update on Storm and Combined Sewer Overflow Management and Treatment, and An Urban Planning Guide for the Assessment of Storm Flow Pollution and the Selection of System Pollution Control Methods." Metcalf & Eddy, Inc., 68-03-2228
P-6	"Use of Polymers to Reduce or Eliminate Sewer Overflows in the Bachman Creek Sewer." City of Dallas, Texas, 11022 DZU
P-7	"Combined Sewer Fluidic Regulator Demonstration." City of Philadelphia, 11022 FWR
P-8	"Development of a Flocculation-Flotation Module." Hercules, Inc., 14-12-855
P-9	"Stormwater Treatment Facilities." City of Dallas, Texas, 11023 FAW
P-10	"The Lawrence Avenue Underflow Sewer System." City of Chicago 11022 EMD
P-11	"Microorganisms in Stormwater." John Hopkins University, 802709
P-12	"Nutrient Removal Using Existing Combined Sewer Overflow Treatment Facilities." Onondaga County, N.Y., 802400

Note: Number appearing in left margin corresponds to reference numbers cited in report text.

Project Reference Number	On-Going Projects
P-13	"Comparison of Alternate Sewer Design." City of Elizabeth New Jersey, 802971
P-14	a) "Refine/Verify a Simplified Model to Handle Large Areas with Minimal Data Input as a Planning Aid." Rochester Pure Water Agency, Y-005141
P-15	b) "Combined Sewer Overflow Abatement Program - Rochester, N.Y." Rochester Pure Water Agency, Y-005141
P-16	"Maximum Utilization of Water Resources in a Planned Community." Rice University, 802433
P-17	"Evaluation of Present Catch Basin Technology and Demonstration and Evaluation of New Upstream Attenuator/Solids Separator Design." Metcalf & Eddy, Inc., 68-03-0274
P-18	"Analysis of Practices for Preparing an Economic Analysis and Determining Infiltration." American Public Works Association, 803151
P-19	"Engineering Aspects of Storm and Combined Sewer Overflow Technology A Manual of Instruction." North Carolina State University, 801358
P-20	"Develop a Movie on Nature/Impacts of Stormwater Pollution As Compared to Other Forms of Water Pollution." (SRO ID No. 61ABR), EPA, Technology Transfer
P-21	"Characterization and Disposal of Combined Sewer Overflow Sludges and Solids." Envirex, 69-03-0242
P-22	"Development and Demonstration of Combined Sewage Treatment Utilizing Screening and Spilt-Air Flotation." City of Milwaukee (Hawley Road) 11020 FDC
P-23	"Demonstration of Screening/Dissolved-Air Flotation as an Alternative to Combined Sewer Separation." City of Racine, Wisconsin, 11023 FWS
P-24	"Sludge Treatment and Disposal Methods for Combined Sewer Overflow." Envirex, 68-03-0242
P-25	"Demonstration Real-Time Automatic Control in Combined Sewer System." City and County of San Francisco, California, 803743
P-28	"Evaluation of Stormwater Treatment Methods." Minnehaha Creek Watershed District, 802535
P-29	"Evaluation and Technology Transfer of the Swirl Concentrator Principle." American Public Works Association, 803157
P-30	"Demonstration/Evaluation of Impregnated Concrete Pipe and Other Methods of Infiltration Control." Texas Water Quality Board, 802651

Project Reference Number	On-Going Projects
P-31	"Trenchless Sewer Construction and Sewer Design Innovation." Sussex County Council, Delaware, 800690
P-32	"The Somerville Marginal Conduit Including Pretreatment Facilities." Boston Metropolitan District Commission, 11023 DME
P-34	"Large Scale Demonstration of Treatment of Storm-Caused Overflows by the Screening Method." City of Fort Wayne, Indiana, 11020 GYU
P-37	"Boston University Bridge Storm Water Detention and Chlorination Station." Boston Metropolitan District Commission, 11023 FAT
P-39	"Ultra-High Rate Filtration of Combined Sewer Overflow and Raw Dry Weather Sewage at Newtown Creek Sewage Treatment Plant." City of New York, 803271
P-40	"East Chicago Treatment Lagoon." East Chicago Sanitary District, 11020 FAV
P-41	"Evaluation of Various Aspects of an Aluminum Storm Sewer System." City of LaSalle, Illinois, 11032 DTI
P-42	"Pilot Plant Studies to Determine the Feasibility of Using High Gradient Magnetic Separation (HGMS) for Treating Combined Sewer Overflows." Sala Magnetics, Inc., 68-03-2218
P-45	"Development of Electromagnetic Flowmeter for Combined Sewer." Cushing Engineering, Inc., 68-03-0341
P-46	"Efficiency of Off-Stream Detention-Retention Measures as Sediment Control Devices." Howard University, 803066
P-49	"Collect and Define Availability of Test Data (Rainfall/Runoff) For Urban Models-Data Base." University of Florida, 68-03-0496
P-50	"Develop and Demonstrate New and Improved Model for Design of Combined Sewers to Prevent Solids Sedimentation and to Optimize Construction Cost." Water Resources Engineers, Inc., 68-03-2205
P-51	"Short Course on Application of Stormwater Management Models-1975." University of Massachusetts, 803069
P-53	"A Guide for Comprehensive Planning for Control of Urban Storm and Combined Sewer Runoff." University of Florida, 802411
P-68	"Verification of Water Quality Impact from CSO using Real-Time Data." County of Milwaukee, 804518
P-70	"Optimization and Testing of Highway Materials to Mitigate Ice Adhesion." Washington State University, 804660
P-71	"Evaluation and Technology Transfer of the Swirl Concentrator Principal." American Public Works Association, 803157

Project Reference Number	On-Going Projects
P-66	"Characterization of Solids Behavior in, and Variability Testing of Selected Control Techniques for Combined Sewer Systems." Northeastern University, 804578
P-67	"Demonstration of Non-Point Pollution Abatement through Improved Street Cleaning Practices." San Jose, California, 804432
P-72	"Demonstration of Erosion and Sediment Control Technology." State of California, 803181
P-73	"Methods of Separation of Sediment From Storm Water at Construction Sites." University of Minnesota, 803579
P-74	"Demonstration and Evaluation of Sediment and Erosion Control Techniques Applicable to the S.E. Piedmont, Fairfield County, South Carolina." University of South Carolina, 803724

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R-1	Total Urban Pollutant Load: Sources and Abatement Strategies: Enviro Control, Inc., for Council of Environmental Quality, Draft Report, October 1973.
R-2	Sources of Metals in New York City Wastewater: Larry A. Klein, et al, JWPCF, Vol. 46, No. 12, December 1974.
R-3	Water Quality Effects From Urban Runoff: Robert E. Pitt and Richard Field, Preprint, 1974 American Water Works Association Conference, Boston, Massachusetts.
R-4	1974 Survey of Needs for Municipal Wastewater Treatment Facilities: USEPA, Office of Water and Hazardous Materials, Washington, D.C.
R-5	Report to National Commission on Water Quality on Assessment of Technologies and Costs for Publicly owned Treatment Works under Public Law 92-500, Volume 1: Metcalf & Eddy, Inc., September 1975.
R-6	Study and Assessment of the Capabilities and Cost of Technology for Control of Pollutant Discharges from Urban Runoff: Black, Crow & Eidness, Inc. and Jordan, Jones & Goulding, Inc., for the National Commission on Water Quality, Draft Report, July 1975.
R-6a	Management and Control of Combined Sewer Overflows: Richard Field and E.J. Struzeski, Journal Water Poll. Control Fed., Vol. 44, No. 6, July 1972, pp 1393-1415.
R-6b	Combined Sewer Overflows: Richard Field, Civil Engineering - ASCE Magazine, February 1973, pp 57-60.
R-6c	Coping with Urban Runoff in The United States: Richard Field, Water Research, Vol. 9, Pergamon Press 1975, pp 499-505.
R-6d	Urban Runoff Pollution Control - State of The Art: Richard Field and John A. Lager, Journal of the Environmental Engineering Division, ASCE, Vol. 101, No. EE1, Proc. Paper 11129, February 1975, pp 107-125.
R-6e	Urban Runoff Must Be Controlled: Richard Field, Baltimore Engineer Magazine, March 1975.
R-6f	Literature Review - Urban Runoff and Combined Sewer Overflow: Richard Field and Pauline Weigel, Journal Water Pollution Control Federation, Vol. 45, No. 6, June 1973, pp 1108-1115.

Note: Number appearing in left margin corresponds to reference numbers cited in report text.

OTHER URBAN RUNOFF POLLUTION CONTROL PROGRAM REFERENCES (continued)

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PLANNED WASTEWATER REUSE
A LITTLE-USED RESOURCE

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ABSTRACT

Water reuse is a common fact of life. Water shortages and the recent recognition in the United States of the need to conserve water has focused attention upon the value of more intentional reuse. Planners recognize the need for a hierachy of water use in the community. All water need not be of the same quality. And the wastewater of a community should be considered a resource.

Wastewater reuse is being specifically recognized by recent legislation. Public Law 92-500, the "Federal Water Pollution Control Act Amendments of 1972", calls for research and facilities construction to permit reuse. Public Law 93-523, the "Safe Drinking Water Act", passed in December, 1974 allows for grants to investigate and demonstrate health implications involved in reclamation, recycling and reuse of wastewater to prepare a safe and acceptable drinking water. The American Water Works Association and the Water Pollution Control Federation has issued a joint statement in support of appropriate reuse.

Municipalities on an annual basis use about $4 \times 10^{10} m^3$ (10 trillion gallons) of water and wastewater return amounts to $3 \times 10^{10} m^3$ (8 trillion gallons). Present wastewater usage for specific purposes such as irrigation and cooling amount to only $5 \times 10^{8} m^3$ (13 billion gallons) or less than 2% of the available flow. Much of the waste flow can be put to productive use.

A great deal of research in EPA is directed toward some facet of wastewater reuse. A major effort is now needed to embark upon a long-term integrated program to permit wastewater reuse for any purpose including to supply drinking water. Some states and major municipalities are already working on a variety of reuse projects. Our combined programs should lead us to a sound base of science and technology that will ensure safety and gain the public confidence for full-scale water reuse. The technology for preparing water of any quality is well advanced. Needed is better knowledge on the level of residues that remain and their possible health effects. Social and economic factors will need additional research.

INTRODUCTION

Abundant supplies of clean surface and underground waters in the United States have been taken for granted until recent years. Severe contamination of many surface supplies has occurred. Increasing instances of groundwater contamination are being found. Thus, our relatively fixed volume of water may become less and less usable. Adequate pollution control measures must be taken and conservation and reclamation of resources must become the rule.

Water has always been used and reused by man. The natural water cycle, evaporation, and precipitation is one of reuse. The return of wastewaters to the streams and lakes of the country is a fact of life. The unplanned reuse of wastewaters is not new. The planned reuse of wastewaters for beneficial purposes has been done in some areas for many years, but it is here that we need to concentrate our efforts for far greater use of our wastewaters.

The quality and quantity of wastewaters produced by the community depend upon such factors as the source of supply, population density, industrial practices, and even the attitudes of the local population. The quality of the environment can be improved by reducing pollution at the source, providing adequate treatment of the wastewaters, and by recycling and reusing wastewater. Public support and some change in social behavior will be required in most instances.

DEFINITIONS

Since there are many different types of wastewater reuse and the term "reuse" has different meanings to different people, the following definitions will be used for this Workshop:

Municipal Wastewater - The spent water of a community, consisting of water-carried wastes from residences, commercial buildings, and industrial plants and surface or groundwaters that enter the sewerage system.

Advanced Waste Treatment - Treatment systems that go beyond the conventional primary and secondary processes. Advanced waste treatment systems may include biological processes, the use of chemicals, activated carbon, filtration or separation by membranes.

Indirect Reuse - Indirect reuse of wastewater occurs when water already used one or more times for domestic or industrial purposes is discharged into fresh surface or underground waters and is used again in its diluted form.

Direct Reuse - The planned and deliberate use of treated wastewater for some beneficial purpose such as irrigation, recreation, industry, prevention of salt water intrusion by recharging of underground aquifers, and potable reuse.

Potable reuse can be further divided into two categories as follows:

Indirect Potable Reuse - The planned addition of treated wastewater to a drinking water reservoir, underground aquifer, or other body of water designed for potable use that provides a significant dilution factor.

Direct Potable Reuse - The planned addition of treated wastewater to the headworks of a potable water treatment plant or directly into a potable water distribution system.

OFFICIAL SUPPORT FOR WASTEWATER REUSE

The role of the U.S. Environmental Protection Agency (USEPA) and its predecessor organizations in wastewater reuse has been stated in various acts. Public Law 87-88 passed in 1961 amending the Federal Water Pollution Control Act directed the Secretary (at that time of Health, Education, and Welfare) "to develop and demonstrate practicable means of treating municipal sewage and other water-borne wastes to remove the maximum possible amount of physical, chemical, and biological pollutants in order to restore and maintain the maximum amount of the Nation's water at a quality suitable for repeated reuse."

This Act gave impetus to the Advanced Waste Treatment Research Program, which began in 1960. The objective of this national program is to conduct research that will develop new and improve existing wastewater treatment processes and ultimate disposal technology, thus permitting maximum removal of contaminants and repeated reuse of the Nation's waters.

Public Law 92-500, the "Federal Water Pollution Control Act Amendments of 1972", recognizes the potentially large benefit to be realized if wastewaters can be renovated for reuse applications. Sections 201 (b), 201 (d), and 201 (g) (2) (B) clearly require 1. that EPA provide for the application of best practicable waste treatment technology, including reclaiming and recycling of water;

2. that construction of revenue producing facilities providing for reclaiming and recycling be encouraged; and 3. that works proposed for grant assistance, to the extent practicable, allow for the application of technology at a later date which will provide for reclaiming and recycling of water. Section 105 (a) (2) authorizes EPA to make grants for demonstrating advanced waste treatment and water purification methods, and Section 105 (d) (2) requires that the Administrator conduct on a priority basis an accelerated effort to develop, refine, and achieve practical application of advanced waste treatment methods for reclaiming and recycling water and confining pollutants.

The Safe Drinking Water Act of 1974 (Section 1444) also contains mandates of importance with regard to renovation and recycling of wastewaters. Section 1444 authorizes a development and demonstration program to: demonstrate new or improved technology for providing safe water supply to the public; investigate and demonstrate health implications involved in the reclamation, recycling and reuse of wastewaters for the preparation of safe and acceptable drinking water.

There exists, therefore, a strong and clear legislative mandate for research development and demonstration of reliable, cost-effective technology for reclaiming and recycling wastewaters for beneficial uses. A major beneficial use is the supplementation of domestic water supplies.

The Water Pollution Control Federation (WPCF) (1) and the American Water Works Association (AWWA) issued a joint resolution that urged the Federal Government to support a massive research effort to develop needed technology. These organizations underscored the "lack of adequate scientific information about possible acute and long-term effects on man's health from such reuse", and also noted that "the essential fail-safe technology to permit such direct reuse has not yet been demonstrated." The resolution recognizes the need for an "immediate and sustained multi-disciplinary, national effort to provide the scientific knowledge and technology relative to the reuse of water for drinking purposes in order to assure full protection of the public health."

The USEPA in a policy statement on water reuse dated July 7, 1972, supports and encourages the development and practice of successive wastewater reuse. EPA does not currently support the direct interconnection of wastewater reclamation plants with potable water systems.

SPECIFIC CONSIDERATIONS GOVERNING REUSE

The reuse of treated effluents is most applicable where large volumes of water are used and the wastes are not highly contaminated. The location of the treatment plant and the possible transport of the renovated water are important considerations. A wastewater renovation plant need not always be located at the same place as the municipal wastewater disposal plant, nor should the renovation process be dependent upon treating the total flow. Treatment processes work most efficiently and economically when dealing with a steady flow of wastewater rather than with the irregular flow normally experienced from urban sources. This condition can be obtained by withdrawing only a part of the urban wastewater; this is depicted in Figure 1, which shows how water renovation and reuse can be planned to best advantage in the community.

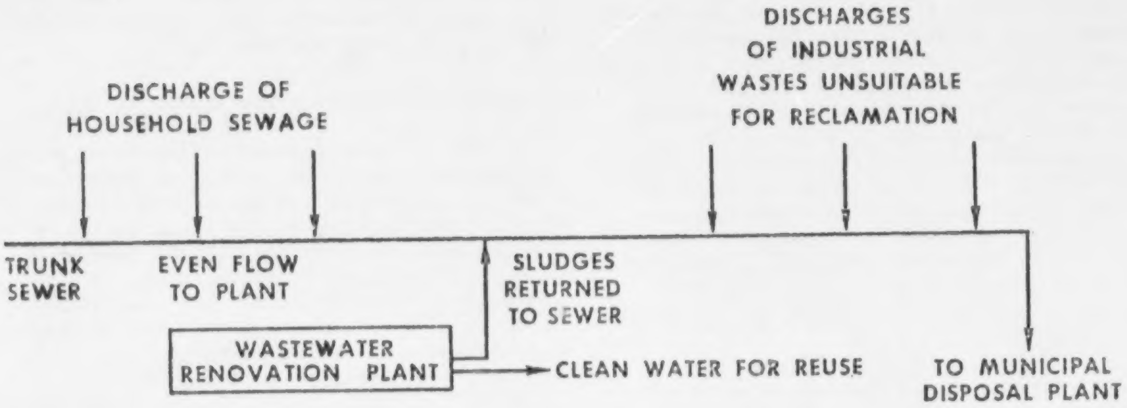
VOLUME OF WASTEWATERS AVAILABLE IN MUNICIPALITIES

Municipalities on an annual basis use about $4 \times 10^{10} m^3$ (10 trillion gallons) of water, and wastewater return amounts to $3 \times 10^{10} m^3$ (8 trillion gallons). Present wastewater usage for specific purposes is shown in Table 1.

As can be seen from the Table, only $5 \times 10^8 m^3$ (136 billion gallons), or less than 2% of the available flow, is used on a planned basis. Much of the waste flow can be put to productive use. Such uses can go a long way in conserving scarce clean water sources.

STANDARDS FOR WASTEWATER REUSE

To ensure the safety of water supplies, standards have to be applied. Standards for drinking water have been available for many years. Although national standards may be set for drinking water, the qualities of river-water, industrial effluents, and reused wastewater are the responsibility of the local controlling authority. Even so, the standards set must take into account


DISCHARGE OF HOUSEHOLD SEWAGE
DISCHARGES OF INDUSTRIAL WASTES UNSUITABLE FOR RECLAMATION
TRUNK SEWER
EVEN FLOW TO PLANT
SLUDGES RETURNED TO SEWER
WASTEWATER RENOVATION PLANT
CLEAN WATER FOR REUSE
TO MUNICIPAL DISPOSAL PLANT

The diversion of wastewater from the trunk sewer to the wastewater renovation plant should be chosen at a point where it is known that the trunk sewer contains only household sewage.

FIGURE 1. Simplified Wastewater Reuse Scheme*

*From World Health Organization Technical Report No. 517 (1973)

TABLE 1. WATER REUSE IN THE UNITED STATES**

Type	Volume in 1971		No. of Plants
	m^3	billion gals	
Irrigation and agriculture	3×10^8	77	338
Industrial	2×10^8	54	14
Recreational	11×10^6	3	5
Non-potable domestic	$< 4 \times 10^6$	< 1	1
*Groundwater augmentation	$< 4 \times 10^6$	< 1	8

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*Estimated from information in EPA publication EPA-660/2-73-006b "Wastewater Treatment and Reuse by Land Application" - Vols. I & II

**From World Health Organization Technical Report No. 517 (1973) the possible transport of pollutants across state borders or the effects of discharges on downstream water users. Standard setting is a most difficult and critical job, with important economic implications. Standards must be given the force of law, and an authority must be created to ensure that they are observed.

Standards governing the quality of water in rivers and lakes are becoming common. Some countries have, and others are formulating, standards applicable directly to effluents, though few countries yet have standards for the planned reuse of treated wastewater. As wastewater reuse grows, it is important that standards be set for specific reuse purposes.

As wastewater - treated or untreated - has been reused in agriculture for a fairly long time, some countries have developed standards for this purpose. A summary of some representative standards for the use of renovated water in agriculture is given in Table 2.

DOMESTIC REUSE

In any reuse application there are a number of points to consider. One very important question is whether the reuse will result in multiple recycle. Multiple recycle produces a buildup of refractory materials, especially inorganic ions, and may require the use of demineralization or other specialized processes. In-plant reuse of industrial water, where actual consumption is small, may lead to a high degree of recycle. On the other hand reuses of municipal wastewater, except for domestic reuse, probably would not lead to multiple recycle. Even in the case of domestic reuse there is not likely to be total recycle. The reason is that less water is ordinarily found arriving at the wastewater treatment plant than is supplied to the municipal water system. Such losses do occur and are quite large in warm, dry areas where domestic reuse is likely to be most widely practiced. In the United States it is estimated (2) that these losses range from less than 20% in humid areas to about 60% in arid areas. Parkhurst et al., (3) point out, based upon experience in the Los Angeles area, that less than 50% of a water supply would be available for reuse. The disadvantage of these large losses is the need for a substantial additional fresh water source. The advantage is that the steady state mineral concentration is reduced. As a result, the degree of demineralization may be reduced substantially below that needed if there were no losses. Also, there is the flexibility of demineralizing either the renovated wastewater or the supplementary water source, there may be advantages to demineralizing the supplementary source.

Another consideration in reuse is the character of the wastewater entering the treatment plant, especially with respect to industrial pollutants. Care must be used to exclude materials that would be detrimental to the reuse application. This is especially true for domestic reuse, but also applies to less sophisticated reuse applications. These materials may not be those usually considered toxic. Ordinary salt brines would be undesirable, for example, if demineralization were being carried out on the renovated wastewater. In Los Angeles County, a survey of the sewer systems has been made to determine how much of the available wastewater has potential for reuse. Waters having heavy metal contamination or high total dissolved solids were considered unacceptable. A similar survey will be necessary for other municipalities planning extensive reuse.

Another point that must be considered is distribution of the renovated water. A multiplicity of piping systems, each one containing a different quality renovated water, will not usually be practical. There may be a number of large consumers in the vicinity of the treatment plant. This would make distribution simple and inexpensive. If the consumers are widely distributed, however, one piping system in addition to the existing municipal water system is almost certain to be the most that will be economically realistic. The result is that the renovated wastewater must be of a quality to satisfy most of the customers without additional treatment. Treatment such as those necessary for boiler water feed would be excluded, since present practice in water supply has shown that those treatments are more appropriately carried out by the user.

INDIRECT REUSE

The following discussion of indirect reuse taken in part from a World Health Organization publication (4) is appropriate.

Examples of indirect reuse in certain rivers:

A study of a number of rivers in the USA, carried out in 1961 (5) showed that at periods of low flow 3.5-18.5% of the water had passed through domestic waste systems. If the volume of industrial effluents is also taken into account, it would be expected that 20-40% of the river water at low flow in some areas may be reused water.

Zoeteman in a World Health Organization publication (6) has plotted the percent of river flow used for community water supply versus the average yearly water flow for 89 rivers, Figure 2. As might be

TABLE 2. EXISTING STANDARDS GOVERNING THE USE OF RENOVATED WATER IN AGRICULTURE*

	California	Israel	South Africa	Federal Republic of Germany
Orchards and vineyards	Primary effluent; no spray irrigation; no use of dropped fruit	Secondary effluent.	Tertiary effluent, heavily chlorinated where possible. No spray irrigation.	No spray irrigation in the vicinity.
Fodder, fibre crops, and seed crops	Primary effluent; surface or spray irrigation.	Secondary effluent, but irrigation of seed crops for producing edible vegetables not permitted.	Tertiary effluent.	Pretreatment with screening and settling tanks. For spray irrigation, biological treatment and chlorination.
Crops for human consumption that will be processed to kill pathogens	For surface irrigation, primary effluent. For spray irrigation, disinfected secondary effluent (no more than 23 coliform organisms per 100 ml).	Vegetables for human consumption not to be irrigated with renovated wastewater unless it has been properly disinfected (< 1000 coliform organisms per 100 ml in 80% of samples).	Tertiary effluent.	Irrigation up to 4 weeks before harvesting only.
Crops for human consumption in a raw state	For surface irrigation, no more than 2.2 coliform organisms per 100 ml. For spray irrigation, disinfected, filtered wastewater with turbidity of 10 units permitted, providing it has been treated by coagulation.	Not to be irrigated with renovated wastewater unless they consist of fruits that are peeled before eating.		Potatoes and cereals—irrigation through flowering stage only.

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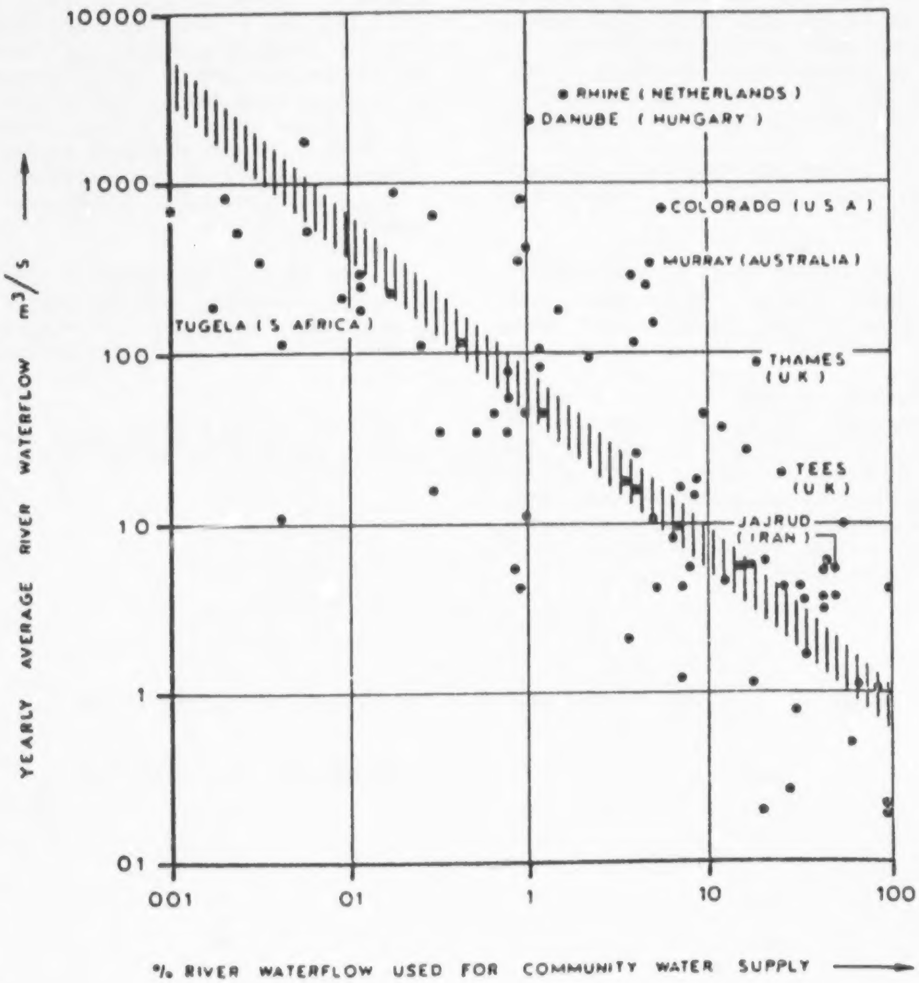
*From World Health Organization Technical Report No. 517 (1973)

Fig. 2 RELATION BETWEEN AVERAGE YEARLY WATERFLOW AND % RIVER WATERFLOW USED FOR COMMUNITY WATER SUPPLY FOR 89 RIVERS IN THE WORLD IN 1968 - 1972



(Zoeteman, 1975)

expected, the rivers with low flow show the greatest percentage use. In the future, the trend-line of Figure 2 will most likely move upward to the higher river flow values. It is important to know the amount of present reuse and to forecast for future reuse. A case in point is the Great Ouse River at Clapham, Untied Kingdom where the present and future content of effluent has been tabulated, Table 3.

The Ruhr River in Germany has a reuse factor of 36% half of the time and has reached 86% under severe conditions. At the 86% concentration of effluent it was reported that 7% of the population of Essen, Germany had non-bacterial gastroenteritis.

As treatment of wastewaters improves future effluents will be less damaging to water quality.

In the United Kingdom, the River Thames, which provides two-thirds of the water supply for the Greater London area, contains about 14% of sewage effluent when flowing at an average rate. During the severe drought of 1975 the flow in the Thames dropped from a daily volume of 35 m^3/s (1,283 ft^3/s) to 2.2 m^3/s (77 ft^3/s) and the flow was, no doubt, nearly all effluent. In times of drought, the water supply source for Agra, India consists almost entirely of partially treated sewage from New Delhi, 190 km away.

The Mardyke is a small river rising to the east of London and discharging into the Thames estuary. It has a dry-weather flow of about 0.2 m^3/s (7 ft^3/s). The Essex River Authority has devised a scheme whereby about 0.4 m^3 (14 ft^3/s) per day of sand-filtered secondary effluent from the Riverside wastewater treatment plant will be pumped into the headwaters of the Mardyke to supplement the flow, instead of being discharged direct to the estuary as at present. The mixture of river water and effluent will be available for abstraction under license by agricultural users in the middle reaches and by industry at the lower reaches-an area now suffering from a shortage of water owing to saline intrusion into the wells that provide much of the supply. A further supply of sand-filtered effluent will be made available for direct industrial use.

The River Authorities in the United Kingdom, according to Billington (7) have formulated a guideline that suggests a maximum limit of 75% sewage effluent in rivers.

In the treatment of polluted rivers, the methods employed at present are based upon

TABLE 3. ESTIMATED CONCENTRATION OF EFFLUENT IN GREAT OUSE RIVER

Year	% Concentration of Effluent
1971	40
1981	60
2001	75
2011	80

those developed over the years for the treatment of relatively unpolluted river water, and it appears that sufficient note may not have been taken of the increasing proportion of wastes in many rivers.

The inadequacy of these traditional methods may perhaps be indicated by the outbreaks of infectious hepatitis in New Delhi in 1955-56 [Dennis (8); Viswanathan (9)], when there were 30,000 cases, and in 1958. The waterworks in question was of modern design and, though there may have been some faults in operation, they were of the sort that may occur at any waterworks. However, at the time of the outbreak drought conditions prevailed, and the water abstracted from the river was estimated to contain about 50% of sullage water.

It appears, therefore, that the public health aspects of the production of potable water from polluted rivers should be reviewed. When rivers contain a high proportion of effluent, the production of water from them should be regarded as analogous to the direct recovery of water from a sewage or industrial effluent, and safeguards appropriate to this situation should be imposed.

There is also an increasing need to consider the optimum distribution of purification between the wastewater treatment plant, the river itself (self-purification), and the treatment plant that produces potable water. There are two extreme cases:

(a) Wastewater is discharged with little or no treatment, all the purification occurring in the river or at the water treatment plant. This practice has been common in the past but is rapidly disappearing. In fact, raw sewage disposal into rivers is prohibited in some countries. Local authorities are requiring secondary treatment and, in some cases, the removal of the nutrients, phosphorus and nitrogen, because incidental pollution that is as yet uncontrolled may well use up the natural purifying capacity of the river.

(b) The wastewater is purified to a standard as high as that of the river water into which it is discharged, so that the type and degree of purification required at the water treatment plant is no different from that which would be required in the absence of the wastewater discharge.

Almost certainly the optimum solution lies somewhere between these two extremes, and optimization studies are required to determine it, taking into account all the costs and benefits involved. This may be difficult in practice because some of the social costs and benefits cannot readily be expressed in economic terms. Such optimization studies are likely to be most successful in the context of a single river basin authority having control over the treatment and discharge of wastewaters and also over the abstraction and treatment of potable waters.

The unintentional reuse of wastewater also occurs widely as a result of the use of river water for agriculture, recreation, and industrial supply and for these purposes, too, there is a need for appropriate safeguards.

DIRECT REUSE

The only known case in the United States where treated municipal effluent is used for a domestic purpose is at the Grand Canyon where the treated effluent is used for toilet flushing and other non-potable uses.

It is not expected, nor is it desirable, that renovated effluents will be piped directly to potable water systems. Some dilution, storage, or other time-holding barrier can usually be provided. In such systems where the interval from effluents to potable waters is short, not only will advanced treatment be given to the effluents but normal water purification procedures will be applied before the water is distributed.

Treated wastewater may be deliberately used in a planned way for a variety of purposes, some of which are shown in Figure 3.

A comprehensive survey of demonstrated reuse technology has been made by SCS Engineers (10).

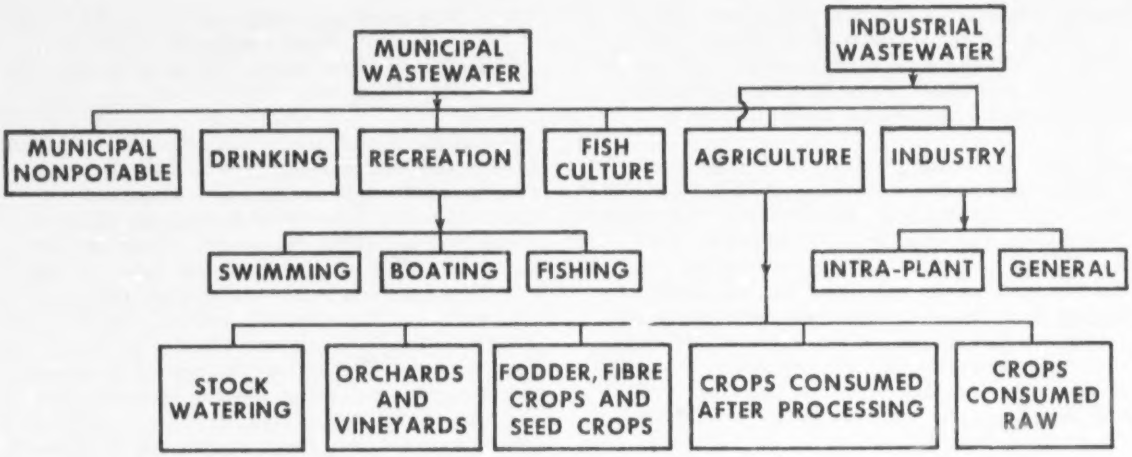


FIGURE 3, Intentional Reuse of Wastewater*

*From World Health Organization Technical Report No. 517 (1973)

REUSE RESEARCH AND OPERATIONS IN OTHER COUNTRIES

Wastewater is reused in many countries. A recent survey by SCS Engineers (10) identified 55 sites where wastewater is reused. By far the largest reuse is irrigation. In some cases raw sewage is used to irrigate grazing lands, vegetables, and grasses. This practice is not generally recommended. A survey of wastewater made by the World Health Organization (4) in 1971 showed that in addition to municipal waters, discharges from breweries, starch factories, textile works, dairy farms, and slaughter houses are used for irrigation purposes. The industrial use of effluents is becoming more common, and recycling within the factory itself is extensive. When effluents are reused in industry, it is common to treat the effluent with chemicals, followed by filtration or other advanced processes depending upon the quality of the water needed. Recreational uses for wastewater exist in the United States and Mexico in the form of artificial lakes, and parks and golf links are watered with treated effluent in a number of countries. As would be expected, the driest countries with growing populations are the most likely to turn to reuse first. An AIChE Symposium (11) contains extensive discussions of foreign reuse projects.

ISRAEL

Israel is an excellent example of a country that is water-short and, by necessity, very active in the wastewater reuse field. Water supply in Israel for agricultural, industrial, and municipal purposes currently requires over 90% of the country's potential water resources. By 1980 there will be a water deficit, and by the year 2000 it will reach major proportions. In 1973 20% of the available municipal wastewater in Israel was utilized for agricultural purposes.

Israel is proceeding with major planned reuse projects, and they look to full development of reuse potential. Even with full reuse Israel will possibly have to resort to desalination of sea water eventually for supplementary supply. A major undertaking that will be watched with great interest is in the Dan Region wastewater utilization project. The City of Tel Aviv, Israel's major city, and six neighboring municipalities have joined together to form the Dan Region Sewage Association to solve their common wastewater disposal problems. For many years the sewage in this area has been going to the sea. In the early 1960's, however, a project was instituted to solve the problem of beach pollution and to provide for the reclaiming and recycling of large volumes of wastewater. The Dan Region project is now in an advanced stage of design and construction. The wastewater will be processed by biological and chemical treatment and then pumped to recharge basins located on the sand dunes south of Tel Aviv. The water will be pumped from the aquifer by a series of recovery wells located several hundred meters away from the recharge basins. Total residence time in the aquifer will be about 400 days. The reclaimed water will be mixed with the groundwater already present in the aquifer. The reclaimed water will then receive some final polishing treatment and possibly further dilution depending upon its intended use, and will be conveyed to the Negev area south of the Dan Region. The reused water will, thus, be a one-pass system and not one of multiple recycling. The initial uses of such water would be for non-potable purposes, but the planners are looking ahead to assuring the possibility of potable use. The cost of reclaiming the wastewater may be as much as $0.32 per m^3 ($1.20 per 1,000 gal), but this amount is not considered excessive in a water-short land.

SOUTH AFRICA

South Africa has long recognized the importance of wastewater reuse, and the country does not expect to have enough freshwater resources by the year 2000. The City of Windhoek, Namibia has already had to resort to the reclamation of municipal wastewater to augment its potable supply. An advanced waste treatment system was installed in 1968 to treat the wastewaters for the city, and up to one-third of the city's water was supplied from the renovated product. Subsequently, the water supply source improved somewhat, and the reclamation plant was shut down. The renovation plant is now being rebuilt on a larger scale, however, and it is fully expected that it will be needed in the future. South Africa has made very good use of municipal effluents for industrial purposes. In nearly all cases, the major use is for cooling but some of the water can be used for steam generation, and the remainder for processing. By careful planning and control, the used water can be adapted for most industrial purposes.

The National Institute for Water Research, located in Pretoria, South Africa, has under operation a 3785 m^3/day (one mgd) pilot plant with many advanced processes for research into improved wastewater treatment. Extensive research related to water reuse is conducted in South Africa. A ten-year study of possible health effects of reuse of wastewater is underway.

UNITED KINGDOM

The United Kingdom has had little need so far to consider very much direct reuse of wastewater. They have, however, made use of such waters industrially. A particularly interesting project has been the development of a direct reuse of sewage effluent in the processing of wool textiles. The wool textile industry has some high water quality requirements, but appropriately treated sewage effluents were found to provide an ample source of sufficiently high-quality water (12).

The Water Research Centre in the United Kingdom in their program plan for 1976/77 list a project relating to assessing the effects of water reuse under present conditions. The elements of their plan are:

1. Conduct an epidemiology study to determine whether there is any relationship between the mortality and morbidity from chronic disease and the degree of contamination from sewage effluents in the drinking water supply.

2. Concentrates from the drinking water of ten towns which have various amounts of sewage effluent in the source will be prepared and the acute toxicity to rats and the mutagenic activity will be determined. A 13-week study will be made on the ten towns and a two- to three-year study will be made on three selected towns.

3. Detailed water quality studies will be made.

AUSTRALIA

Australia is regarded as a dry continent and the concentration of populations causes some water resource problems. The City of Melbourne has made unique reuse of the city's wastewater by irrigation of otherwise barren land to graze large numbers of cattle and sheep. A net income of $0.5 million was earned from the livestock oepration in 1973-74. The water management approach of Melbourne is described by Croxford (13). The Ministry of Water Supply of the Victorian State Government, Melbourne, Australia established a Reclaimed Water Committee in the early 1970's and are preparing a master plan for reuse for high quality purposes by the year 2000.

MEXICO

Large areas of Mexico are arid or semi-arid. Wastewater reuse is vital in these areas. Several industries in Monterrey are supplied with chlorinated activated sludge effluent. The plants then treat the effluent further for the specific use required. In Mexico City, Chapultepec Park uses treated wastewater effluent for irrigation and to supply water to two lakes. Several other areas of the city use effluents for irrigation.

JAPAN

Japan is well aware of the value of water and the possible reuse of wastewaters. According to Ikehata (14) the use of regenerated wastewater for potable matter is not likely in Japan and even some non-potable use has met with some citizen objections. Industrial reuse is acceptable.

In 1973 Japan established a Water Reuse Promotion Center (3-4, Akasaka 2 chome, Minato-ku, Tokyo 107). The objective of the Center is to promote the development and practical application of water reuse technology, by reclamation of wastewater and desalination of sea water in order to cope with the very acute water demand in Japan. The main activities are as follows:

1. Construction and operation of test plants for water reuse
2. Promotion of the research and development of water reuse technology
3. Consulting service on construction of water reuse plants
4. Training of water reuse engineers
5. Establishment of standards for water reuse facilities and certification of their performance
6. Information service about water reuse technology
7. Survey on water reuse
8. Promotion of international cooperation in the field of water reuse

CURRENT AND PLANNED RESEARCH IN WASTEWATER REUSE

A. EPA

Research programs in the past within the EPA, and predecessor agencies, have not often been directed toward wastewater reuse as such. However, many of the research projects conducted have developed data needed for assessing the various aspects of reuse. The former Advanced Waste Treatment Research Program (now the Wastewater Research Division) in seeking to remove the maximum amounts of contaminants from water contributed data needed for reuse considerations. Within the last two or three years more direct attention has been given to reuse programs. Some EPA projects that impact reuse are as follows:

Municipal Environmental Research Laboratory

Wastewater Research Division

Over a period of several years projects in this Division related to reuse are listed below. Nearly all were completed in 1976.

1. "Heavy Metal and Virus Removal in AWT Systems at the Blue Plains Pilot Plant"

   Start Date: 7/71

   Completion Date: 6/76

2. "National Wastewater Reuse Research Needs Workshop"

   Start Date: 11/75

   Completion Date: 10/76

3. "Water Reclamation Project for Antelope Valley, California"

   Start Date: 8/70

   Completion Date: 4/76

4. "Heavy Metal and Virus Removal in a Tertiary Pilot Plant Employing Nitrification, High Lime, Filtration, Carbon and Chlorination"

   Start Date: 2/72

   Completion Date: 6/76

5. "Evaluate and Document Results of Lime Coagulation for Removal of Virus from Municipal Sewage"

   Start Date: 8/69

   Completion Date: 4/76

6. "Complete State-of-the-Art on the Reuse of Treated Wastewater (Part I) - Complete Addendum Report on Reuse for Groundwater Purposes (Part II)"

   Start Date: 6/72

   Completion Date: 6/75 Part I
   4/76 Part II

7. "Evaluation of Full Scale Wastewater Reuse System for Domestic Groundwater Replenishment"

   Start Date: 1/76

   Completion Date: 12/78

8. "Disinfection and Color Removal from Secondary Effluent in a 10 GPM Ozone Pilot Plant"

   Start Date: 7/73

   Completion Date: 4/76

9. "Characterization for Potable Reuse and Ultraviolet Disinfection of Municipal Effluents"

   Start Date: 6/74

   Completion Date: 6/76

10. "Demonstration of Tertiary Physical-Chemical Treatment for Production of Cooling and Irrigation Water" Colorado Springs -

    Start Date: 6/68

    Completion Date: 6/76

11. "Evaluation of AWT Systems at Blue Plains Pilot Plant for Potable Reuse Purposes"

    Start Date: 10/74

    Completion Date: 10/76

12. "Measure Parameters of Potable Water Quality Including Concentration of Organics for Health Testing in Effluents from AWT Systems, and Improve Concentrating Techniques"

    Start Date: 6/74

    Completion Date: 9/75

13. "Economic Analyses of Water Supply Systems to Evaluate Relationships Between Cost of Developing New Water Supplies and Wastewater Renovation for Potable Reuse"

    Start Date: 6/74

    Completion Date: 6/76

14. "Wastewater Reclamation Project, St. Croix, U.S. Virgin Islands"

    Start Date: 6/74

    Completion Date: 6/76

Economics of Reuse

Pricing of water and all of the costs that go into the acquisition, purification, power and pumping, transmission and distribution and support services is vastly complicated. Subsidies and variable bookkeeping practices add to the difficulty of finding the true cost of water.

An EPA project (15) has recently been completed which attempts to make an economic evaluation of renovated wastewater as an alternative source for municipal water supply. Eleven utilities were investigated. Detailed studies were made of the Dallas, Texas system and the San Diego, California system. Both cities are in water-short areas and both will have to supplement their present water supplies in the future. Conclusions are that providing up to 50% of renovated water for the two cities would cost from 25 to 50% more than further development of natural supplies.

Such a conclusion in no way indicates that water renovation is not a viable method for supplementing water supplies or would not be different in other cities. The EPA project is valuable in demonstrating techniques of cost standardizing for making comparisons of natural and renovated waters. Reuse at the 50% level in the two cities extends the water supplies significantly but even with reuse early in the next century, more water would be needed.

A very useful report by Schmidt and Ross (16) can serve as a guideline to communities and planners dealing with reuse. Cost effectiveness analysis is necessary. The authors provide check lists and case histories to help a community analyze the feasibility of reuse. To determine if reuse is potentially practical, the check list shown below is used. If one or more is true for a given area, then an economic analysis should be performed.

Checklist for Determining if Wastewater Reuse is Potentially Practical

1. Existing or future fresh water supply is limited.
2. Existing or future fresh water supply is relatively expensive.
3. The area presently includes or will include individual entities who use high volumes of water.
4. Municipal wastewater of relatively high quality is presently discharged for disposal.
5. Requirements for improved wastewater effluent are impending or are anticipated.

Application of the analysis for reuse was conducted at Santa Barbara, California and Hampton Roads, Viriginia. At both locations feasibility and cost effectiveness was shown for reuse of, at least, part of the municipal effluent.

Water-short areas of the world may not have a choice - they will have to reuse wastewaters.

Major Issues Relating to Reuse

There are a number of logical questions to be asked and issues to be solved before planned wastewater reuse will be used to the maximum extent. Some of these issues are: A. Adequacy of Treatment Technology; B. Health Effects; and C. Socioeconomic Factors.

A. Adequacy of Treatment Technology

Dean and Convery (17) have analyzed the status of wastewater treatment technology for preparing potable water. They concluded that:

1. Wastewater treatment performance data is not available for all of the parameters included in existing drinking water standards.
2. Information that is available indicates that wastewater treatment technology is capable of meeting all the standards that have been measured.
3. Full-scale wastewater treatment facilities can be operated reliably over long periods of time.
4. Long-term reliability testing of a full-scale facility specifically designed to produce potable quality water is needed.

On a practical basis, known treatment technology is adequate for non-potable reuses. Many people rightfully have reservations about reuse for potable purposes. These reservations usually refer to a direct treated wastewater to drinking water connection. While this type of direct interlock will seldom be needed it represents our most critical technology goal. Storage and dilution prior to use is more acceptable.

Present day AWT systems can produce a water from municipal wastewater effluents that, by drinking water quality standards, is as good as the drinking water. The EPA work at the Blue Plains pilot plant has produced water very close in quality to the Washington, D.C. tap water. These highly treated waters may have small residues as follows:

Total Organic Carbon	1 - 2 mg/l
Total Nitrogen	1 - 2 mg/l
Chlorinated Organics	1 to 100 µg/l

The meaning of these small residues is not known and is the subject of ongoing research.

Proper disinfection of AWT waters free of solids and preferably of very low turbidity (0.1 unit) can assure safety from microbiological contaminants and viruses.

B. Health Effects

A fundamental problem which retards wide-scale application of wastewater reuse is the lack of suitable standards. Current drinking water standards are not considered applicable to reused waters because of the historic concept of using the best properly protected sources of water available. Some standards have been issued for use of effluents in agriculture. California has been a leader in issuing standards.

There are, of course, more uncertainties in using wastewaters as a source for a potable water because of the wide differences from place to place and the lack of control of spills, additives, and miscellaneous discharges to sewer systems.

Adequate health effects data of trace contaminants in present drinking waters and reused waters is a major block to potable reuse. Chemicals are, generally, of greater concern than the microbiological contaminants. The concern for chemicals has been accentuated recently with the discovery of asbestos, chloroform (from disinfection with chlorine), vinyl chloride, and a host of organic chemicals present in the microgram per liter range of concentrations. Carcinogenic properties are ascribed to many of these chemicals. Little has been done in the past on the health effects of water contaminants. This gap in information is now beginning to receive research attention. Development of a data base in the health field will be slow and costly.

C. Socio-economic Factors

The socio-economic aspects of large-scale purposeful reuse of renovated waters has received only minor attention. Particular factors involved in socio-economic issues include: public acceptance, problems of cities pioneering reuse, conflicts between water and wastewater agencies, implications to downstream users, and energy considerations. Milliken and Creighton (18) conclude that the principal question with regard to recycled water is whether it can be efficiently and equitably integrated into the socio-economic and social framework of the existing systems. They recommend that where reused water can be made cost competitive it should be substituted for conventional water at levels furtherest from human contact. This will conserve present supplies and allow the lead time needed to resolve socio-economic problems.

Research Needs

In 1975 the EPA, along with the American Water Works Association and the Water Pollution Control Federation, sponsored a workshop entitled "Research Needs for the Potable Reuse of Municipal Wastewater." The proceedings of this workshop are available (19). These proceedings are a valuable compendium of up-to-date information on wastewater reuse as well as a guide to research needs.

The principal conclusions from the workshop are:

General

1. The identified needs pertain to indirect wastewater reuse as well as the possible potable reuse.

2. Identification of contaminants in present drinking waters and the development of optimum control processes is urgent.

3. Develop rapid, sensitive, and accurate testing procedures for all types of contaminants and seek on line continuous monitoring where required.

4. Toxicological studies using in vitro and screening and testing are of high priority.

5. Long-term health effects of reuse involving retrospective and prospective epidemiologic surveys of populations exposed to indirect wastewater reuse should be undertaken as an international collaborative activity.

6. Improved exchange of information relative to reuse is needed here and abroad. One coordinating body in existence is the World Health Organization International Reference Centre for Community Water Supply located at The Hague.

The six separate workshop groups elaborated upon the specific needs in the area assigned.

UNITED STATES REUSE RESEARCH PROGRAMS OUTSIDE OF EPA

The U.S. Office of Water Resources Research has funded several studies to determine whether the public will accept the use of renovated water for various purposes. This type of information ties directly to the EPA programs, as the findings suggest that the public will accept renovated water for a municipal supply if they have knowledge that the water is safe.

The U.S. Army is carrying out research on using renovated water for potable purposes, as the Army may have special applications for this approach in certain geographical areas. The MUST (Military Unit, Self-contained Transportable) field hospital project is researching the limited reuse potential of all hospital liquid wastes (not human excrement). Use of ozone, carbon adsorption, and reverse osmosis are key elements in the process. The EPA is serving in an advisory capacity to the project. Much of this research is by contract. In addition, the Army is developing plans for a $7.6 million pilot plant at Washington, D.C. to study treatment technology and related health effects when the Potomac River estuary and the Blue Plains sewage treatment plant effluent are recycled as a supplemental water supply. This has been authorized under P.L. 93-251. The EPA will serve in an advisory capacity for the pilot plant work. The facility is scheduled for operation by January, 1979.

Studies have been made or are planned on using renovated water for potable purposes at the local level. The City of Dallas, Texas operated a pilot plant to study the feasibility of renovated water. The EPA furnished an engineer to work at the pilot plant for several years. A report on this effort including treatment technology and effluent characteristics should be available in early 1976. The City of Denver just completed preliminary plans for its one million gallon per day pilot plant for direct recycle of wastewater. The EPA is cooperating with the City in developing its plans. With this facility, Denver will develop design data for its 100 mgd plant.

The World Health Organization International Reference Centre for Community Water Supply is initiating certain research studies related to wastewater reuse for potable purposes. They have asked that EPA participate in these international programs. Some countries have more pressing water supply problems than the United States. An EPA research program would have international impact and reuse research in other countries would, certainly, have implications in the United States.

Reuse research in California is a major activity. Details are covered in a separate paper in this Conference.

Research Coordination

The Research Foundation of the American Water Works Association is serving as a coordinating and reporting agency for research being conducted in the area of water reclamation for potable use. The contract is with the following agencies and distribution of information is to the contractors.

California Association of Reclamation Entities of Water - San Jose, CA.

Dallas Water Utilities - Dallas, TX.

Board of Water Commissioners - Denver, CO.

Fairfax County Water Authority - Annandale, VA.

Department of Water and Power - The City of Los Angeles, CA.

County of Nassau, Department of Public Works - Mineola, NY.

Department of the Army, U.S. Army Medical Bioengineering Research and Development Laboratory - Fort Detrick, MD.

U.S. Environmental Protection Agency - Cincinnati, OH.

The World Health Organization sponsored a meeting in The Hague in 1975 on "Health Effects Relating to Direct and Indirect Reuse of Wastewater for Human Consumption." One of the recommendations of the meeting was to designate the WHO-sponsored International Reference Centre for Community Water Supply located at The Hague as International Coordinating Agency for the Study of Health Effects of Direct and Indirect Reuse of Wastewater for Human Consumption.

RESPONSIBILITIES TO PROMOTE REUSE

Water reuse, if we are to achieve the maximum benefits, has to be planned on a broad basis. How might the responsibilities for optimum reuse be distributed? The Federal role has been outlined earlier in this paper. Let us consider the role of the local agency and industry.

Local Agencies Role in Water Reuse

1. Promote to the extent possible, consistent with water quality considerations, the renovation and reuse of wastewaters.
2. The water reuse potential on an area-wide basis should be determined. Specific studies on reuse in sub-areas would follow. Examples of objectives include the following:
    a. Determine a ranked order of water uses in the local area in accordance with the acceptability of different levels of water quality.
    b. Demonstrate how water reclamation can supplement available water supply.
    c. Demonstrate how wastewater reclamation can relieve downstream collection and treatment systems.
    d. Develop legislative, economic and planning procedures to implement water reuse.
    e. Examine all available technology for reuse, assess public acceptance, aesthetics, and cost-benefit ratios.

Industrial Role in Water Reuse

1. Examine all plant practices to obtain optimum water use and recycle with a minimum of waste effluents. Try to become a minimum discharge industry.
2. Consider the use of available waters of secondary quality (sewage plant effluents, brackish waters, sea water, etc.) for suitable plant purposes.
3. Examine chemicals and processes contributing to pollution and determine if changes in either will help abate pollution.
4. Consider product recovery from waste materials.
5. Be aware of the new developments in processes to control pollution.
6. Make all plant personnel water and pollution conscious.

TIME TABLE

The United States is in the fortunate position of not needing to immediately turn to wastewaters as a direct source for preparing potable waters. Other countries such as South Africa and Israel have more urgent needs, and we shall learn from them. We all know of the long time needed to bring research results into practical full-scale use. Our time to develop and prove full-scale reuse is short. Our needs for reuse will be much greater in 10 years, and in 20 years it will be a necessity in many parts of the United States and around the world. It is the responsibility of engineers and scientists around the world to provide the scientific and technological base to permit reuse for all purposes.

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INDUSTRIAL WASTEWATER PRETREATMENT AND JOINT TREATMENT IN POTW

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ABSTRACT

The authority for guidelines and pretreatment standards, effluent limitation of industrial wastewaters, and the National Pollutant Discharge Elimination System (NPDES) are discussed. The U.S. Environmental Protection Agency (EPA) policy regarding pretreatment and the advantages vs. disadvantages are cited including sludge disposal considerations. Various essential institutional relationships between Federal, state, municipal, industrial and the elements of pretreatment are presented in tabularized form. Interference with the operation of a Publicly Owned Treatment Works (POTW) system by industrial wastewater components are discussed. Included are materials which inhibit biological treatment processes. Synergistic and antagonistic effects of combinations of pollutants are also presented. In addition to these chemical factors, the significant biological factors associated with acclimatization for joint treatment are considered. Objectives of the Office of Research and Development joint industrial-municipal program are presented along with a glossary of several new terms currently associated with this developing technology.

INTRODUCTION

Title III of PL 92-500, "Standards and Enforcement," specifically refer to the pretreatment of pollutants introduced into Publicly Owned Treatment Works (POTW). Section 304(f) which states:

> "For the purpose of assisting States in carrying out programs under Section 402 of this Act, the Administrator shall publish . . . and review at least annually thereafter and if appropriate, revise guidelines for pretreatment of pollutants which he determines are not susceptible to treatment by publicly owned treatment works. Guidelines under this subsection shall be established to control and prevent the discharge into the navigable waters, the contiguous zone, or the ocean (either directly or through publicly owned treatment works) of any pollutant which interferes with, passes through, or otherwise is incompatible with such works."

These guidelines should not be confused with pretreatment standards for both existing and new sources, promulgated under the authority of Sections 307(b) and 307(c) respectively. Section 307(b) is directed toward existing sources. By authority of Section 307, the EPA promulgated (November 8, 1973) general pretreatment standards. Pretreatment standards set forth rules and regulations designed to protect POTW's from the harmful industrial wastewaters. This Section also authorizes development of pretreatment standards for specific major industrial categories of non-residential contributors, and pretreatment performance standards for new sources.

The distinction between pretreatment standards and pretreatment guidelines must be emphasized. Guidelines only recommend approaches to developing pretreatment policy. Pretreatment standards represent rules and regulations which are enforceable by the Federal Government. (Figure 1)

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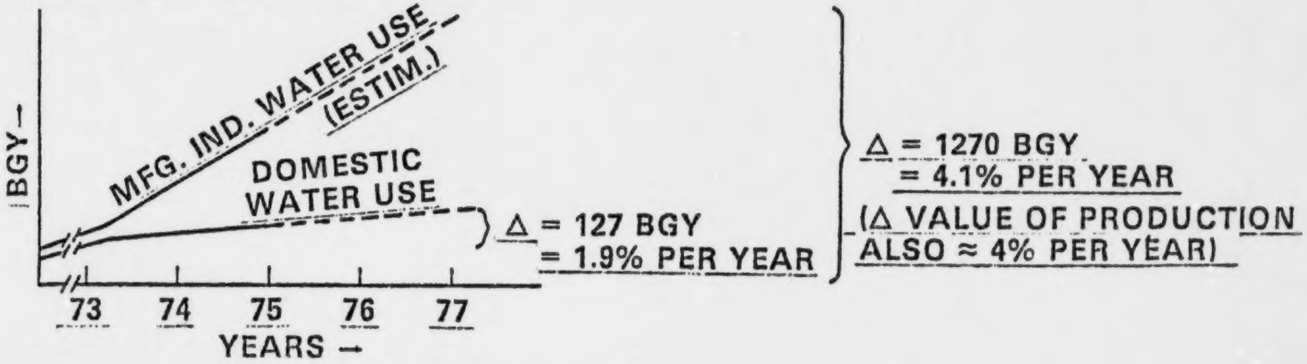
I. MAGNITUDE

CONTROLLABLE POINT SOURCES	WASTEWATER* 10^9 GPY	BOD 10^6 LBS/YR	SS 10^6 LBS/YR
ALL U.S. INDUSTRIES	14300	23000	20500
MUNICIPALITIES	5120	8700	10200
ELECTRIC UTILITIES	36700	-	-

II. COMPLEXITY

	HEAT	OD	SS	TDS	TOXICITY	COLOR	NUTRIENT
INDUSTRIAL WASTES	✓	✓	✓	✓	✓	✓	✓
MUNICIPAL SEWAGE	N/A	✓	✓	N/A	N/A	N/A	✓
ELECTRIC UTILITIES	✓	N/A	✓	✓	N/A	N/A	N/A

III. RELATIVE PROBLEM GROWTH RATE



*Dept. of Commerce, Bureau of Census, 1975

Figure 1. THE PROBLEM

FEDERAL PRETREATMENT STANDARDS

EPA issued (November 8, 1973) general standards for pretreatment of pollutants introduced into POTW's. Subsequently, the Agency has proposed and promulgated numerous pretreatment standards on specific industry category wastewater discharges for both existing and new sources. Numerous results of these proposed and promulgated regulations have indicated that pretreatment procedures, both for the establishment of and for the enforcement of pretreatment standards, and the pretreatment standards themselves were not clearly understood by the general public. Therefore, EPA decided to clarify and simplify the existing pretreatment standards. The proposed standards will be published in the Federal Register. Then the proposed regulation and an appropriate discussion will be incorporated in these guidelines.

The pretreatment standards are intended to be national in scope. States or municipalities will supplement the Federal standards with additional pretreatment requirements taking into account local conditions. Water quality standards reflected in the POTW's NPDES permit or characteristics of the treatment process may necessitate more restrictive pretreatment requirements. The Federal pretreatment standards do not preclude municipalities and States from establishing pretreatment standards of their own, not in conflict with any Federal pretreatment standard. State or local standards may be necessary to control types, flows, concentration and variability of industrial and commercial discharges into municipal treatment works.

Occasionally the Federal pretreatment standards will not protect the operation of the POTW; as when the quantity of a pollutant not susceptible to treatment in a POTW from a large contributing industry, would result in a concentration of the pollutant in the influent to the treatment works and inhibit its performance. Then the municipality would have to supplement the Federal standards. Pretreatment of wastewaters is necessary for spill protection or flow equalization to ensure compliance with the Federal pretreatment standards and permitted effluent limitations.

The pretreatment standards published for the first fifteen industries do not establish limitations on pollutants contributed to municipal systems because the pollutants from these industries are usually susceptible to treatment in a POTW. For the first fifteen industrial categories there is no specific numerical pretreatment requirement at the Federal level. (Figure 2.)

EFFLUENT LIMITATIONS FOR POTW'S

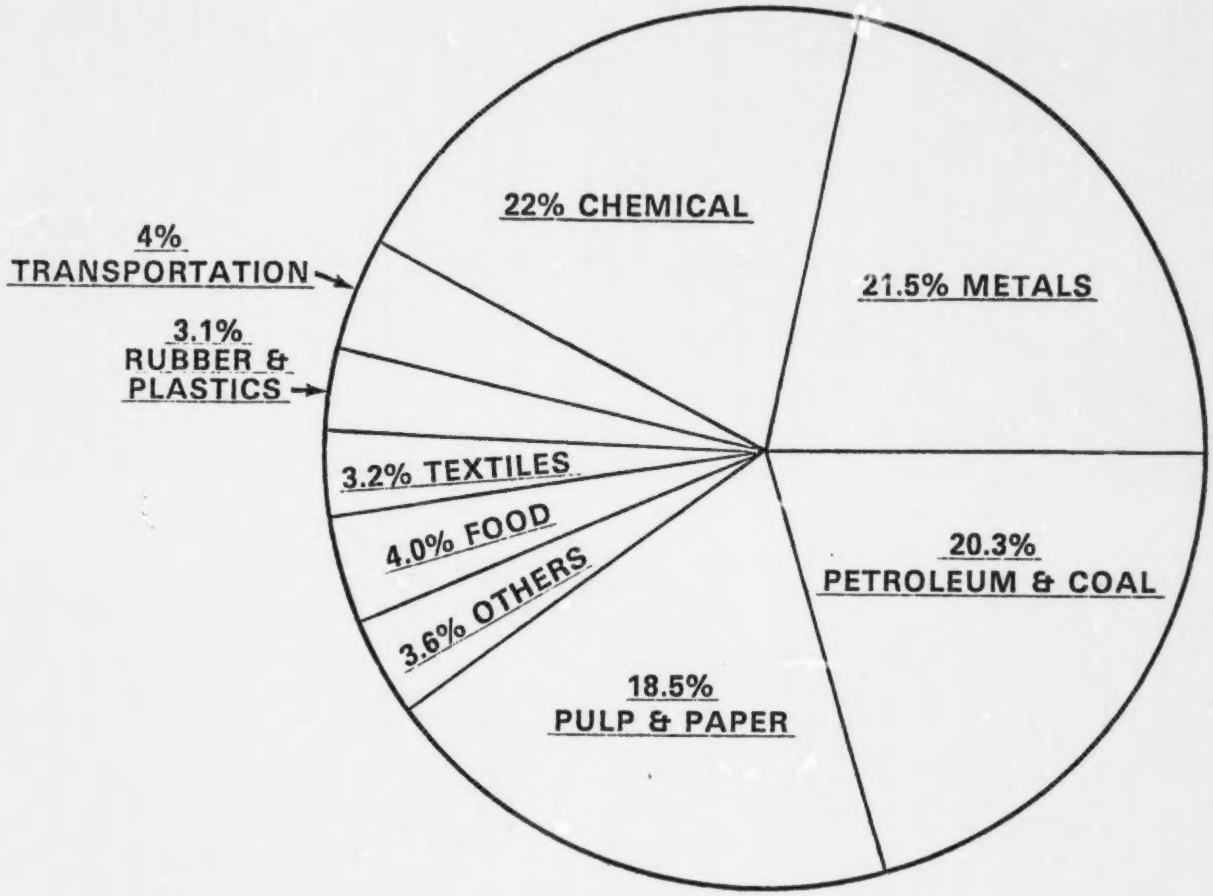
POTW's are required to obtain permits for their discharges and to meet certain minimum effluent standards. Both POTW permits and effluent standards must have a direct bearing on the control of industrial pollutants that must be undertaken within a particular system.

Most POTW's discharge their effluent to navigable waters and the Act requires that effluent limiting regulations be promulgated. Limits are on concentrations of BOD suspended solids, fecal coliform bacteria, and pH range. The pollutants are generally susceptible to treatment in POTW's. Therefore, industrial pollutants become important to secondary treatment standards when the contribution causes these pollutant discharge limitations to be exceeded.

Treatment requirements to meet water quality standards are normally in the NPDES permit. Control of applicable industrial pollutants can be important in helping the municipality meet its NPDES permit requirements. Limitations on a POTW, beyond standard secondary treatment, are for controlling the effects of non-domestic wastewaters.

The NPDES permit impacts on State or local pretreatment policy. First, as a part of the permit application, the permittee must obtain preliminary information on the activities and wastewater characteristics of major industrial contributors within the collection system. Effluent limitations for pollutants not susceptible to treatment in a POTW may be individually established by the permitting agency.

Secondly, under the Federal NPDES permit regulations (40CFR126) the permittee is required to provide notice to the Regional Administrator of the following:



SOURCE U.S. CENSUS OF MANUFACTURES

Figure 2. Industrial Water Use

(i) Any new introduction of pollutants into such treatment works from a source which would be a new source as defined in Section 306 of the Act if such source were discharging pollutants;

(ii) Any new introduction of pollutants which exceeds 10,000 gallons on any one day into such treatment works from a source which would be subject to Section 301 of the Act, if such source were discharging pollutants; and

(iii) Any substantial change in volume or character of pollutants being introduced into such treatment works by a source introducing pollutants into such works at the time of issuance of the permit.

This notice must include information on the quantity and quality of wastewater introduced by the new source into the publicly owned treatment works, and any anticipated impact on the effluent discharged from such works.

POLICY

Pretreatment

Most States or municipalities will require pretreatment to comply with the effluent limitations in the NPDES permit. Pollutants which would interfere with or pass through the POTW, resulting in a violation of any of these NPDES permit requirements, must be pretreated or rejected from the system. Pretreatment is most commonly required for incompatible pollutants to prevent interference with treatment processes or pass through to receiving waters.

Where design capacity is not available pretreatment for compatible pollutants may be necessary to comply with NPDES permit effluent limitations.

Pretreatment of incompatible wastes offers several operational advantages to POTW's. One significant advantage to the municipality is the specialized treatment that each wastewater contribution receives, and the fact that the potential for plant upset is greatly reduced by pretreatment.

Joint Treatment

Joint treatment is an alternative that can be advantageous to both the POTW and the industry. Treatment of industrial wastewaters in a POTW is incidental to its primary function of treating domestic sewage. Where the industrial contribution constitutes a significant portion of the total flow and substantially alters the concentration of pollutants normally contained in domestic sewage, the public agency may resort to the joint treatment approach. Then the industry or industries contributing the pollutants is made a partner in the design and construction of the system, and the treatment works are designed to specifically remove the industrial pollutants. Both capital costs and operating costs are allocated to the industry and the public agency according to an agreement arrived at through negotiation, or as required by Federal regulations if construction grant funds are involved.

Joint treatment of industrial wastewaters with municipal domestic sewage offers these advantages:

- Savings in capital and operating expenses due to the economics of large-scale treatment facilities
- Increased flow which can result in reduced ratios of peak to average flows
- More efficient use of land resources, particularly in cases where available land for treatment facilities is scarce
- Improved operation (larger plants are potentially better operated than smaller plants)
- Increased number of treatment modules with resultant gains in reliability and flexibility
- More efficient disposal of sludges resulting from treatment of wastewaters containing pollutants susceptible to treatment in POTW's

- Utilization of the nutrients available in domestic wastes for biological treatment of industrial wastes which are nutrient deficient.

Possible disadvantages of joint treatment are as follows:

- Where the pollutants are different from those usually treated in a POTW design to treat the combined industrial/domestic waste stream for these pollutants may not be cost-effective

- Joint treatment by definition implies that the POTW was designed so as not to be interfered with by industrial wastes. However, where this requires design modifications ordinarily not required for domestic wastes, joint treatment may not be cost-effective

- If joint treatment results in sludge disposal or utilization problems, it may not be acceptable

- Some costs for the construction of joint treatment works solely to treat industrial pollutants are not eligible for Federal construction grants.

Sludge Disposal

The ultimate disposal of sludges produced by either pretreatment or joint treatment operations is an important factor to consider. The POTW must be aware of the effect on environmental problems that may result from sludge disposal.

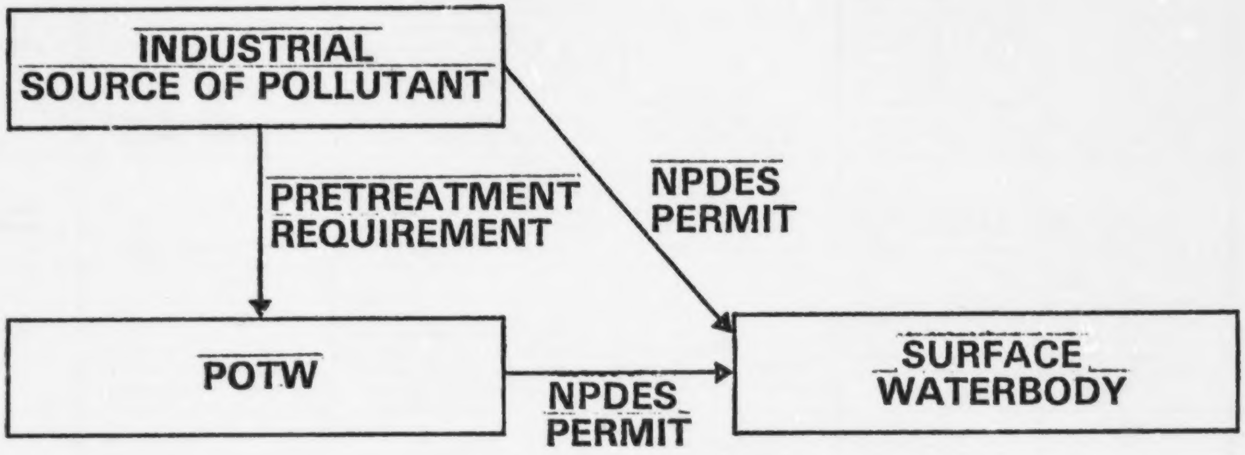
Pretreatment facilities normally remove incompatible pollutants that may be deposited in the POTW sludge. This can be an advantage in terms of the environmental effects of the ultimate disposal of sludge. Incompatible pollutants in sludges can cause problems in most disposal techniques utilized, including incineration, landfills, ocean dumping, and land spreading. consequently, the removal of incompatible pollutants at their source by pretreatment is advantageous.

Incompatible pollutants removed by pretreatment still require an ultimate disposition. The sludge produced by pretreatment operations may be a source for by-product recovery or recycle. When this is not economically or technically feasible, disposal of sludge is necessary. Although the sludges produced by industrial pretreatment may not technically be under municipal regulatory control, the impact on other environmental areas should be noted.

Institutional Relationships

Figure (3) places pretreatment requirements on the vector between industrial point sources and the publicly owned waste treatment works. It shows that in part, pretreatment requirements are indirectly affected by the NPDES requirements of the POTW.

The various pretreatment elements and the institutional relationships are best shown on TABLE 1.



ALTHOUGH THE NPDES DOES NOT REQUIRE ISSUANCE OF PERMITS TO INDUSTRIES DISCHARGING TO SEWERS*, THE POTW THAT TREATS THE WASTE IS REQUIRED TO HAVE A DISCHARGE PERMIT. AS PART OF THE NPDES PERMIT FOR THE POTW, THE MUNICIPAL OPERATING AUTHORITY MUST SUBMIT FORMS IDENTIFYING THE DISCHARGES TO THE SEWERS FROM MAJOR CONTRIBUTING INDUSTRIES

Figure 3. Industrial Point Source Control

TABLE 1. INSTITUTIONAL RELATIONSHIPS

PRETREATMENT ELEMENTS	FEDERAL	STATE & AREAWIDE PLANNING/MANAGEMENT	STATE	MUNICIPAL	INDUSTRY
REGULATORY AUTHORITY	Specified by Sec 307(b) & (c) of PL 92-500. Regulations establish pretreatment standards for significant industrial users. Federal guidelines on pretreatment are specified in Sec 304 (f).	Reviews POTW Permit, and 201 plans. Creates 20 year plan. Establishes regulatory program to insure industrial pretreatment requirements are met.	May give permit to POTW and specify numerical limits. Establishes enabling law for municipal authority.	Sets local PT Standards. Defines Service Area.	None
ENFORCEMENT	Direct enforcement of Federal PT Standards	Designates agency to handle enforcement.	May have direct enforcement over POTW Permits compliance; in some cases over sewer dischargers into POTW.	Major enforcement authority over industrial users of sewers.	None
MONITORING	May carry out monitoring of POTW discharges. Reviews POTW monitoring data. Access and inspection rights.	May review POTW monitoring data; may review self-monitoring data from industrial users of POTW.	May carry out monitoring of POTW discharges. Reviews POTW monitoring data. Access and inspection rights.	May monitor: -influent to POTW -effluent -industrial users (access and inspection).	May perform self-monitoring. (most common).
FINANCIAL	Construction grants subject to POTW NPDES review.	Reviews and approves federal construction grants. May provide an assessment of economic impact of pretreatment requirements on local industry.	May provide "matching funds" for construction grants	Establishes user charges, surcharges in order that industry contributes equitably to capital and O&M costs of POTW.	Subject to user charges and fines. Cost recovery requirements. Assume cost of monitoring.
PRETREATMENT STANDARDS	Federal minimum Standard setting.	Incorporates federal and local standards inWQM plan. Aids municipality in preparation of PT ordinance standards.	Sets State standards in compliance with federal minimum standards.	Sets local standards in compliance with federal minimum.	Subject to federal and local standards.
RESIDUALS	Provides guidance on sludge disposal.	Identifies both toxic components of sludge and industrial contributors. Proposes standards for sludge quality and specifies pretreatment necessary to meet sludge standard. Establishes management program for disposal of industrial pretreatment sludges.	Specifies and approves sludge disposal options.	Carries out sludge disposal in an environmentally acceptable manner. Coordinates with 208 agency in selecting sites for disposal of industrial pretreatment sludge.	Generates pretreatment sludges.

PT = Pretreatment

POTW = Publicly Owned Treatment Works

TABLE 1. (Continued) INSTITUTIONAL RELATIONSHIPS

PRETREATMENT ELEMENTS	FEDERAL	STATE & AREAWIDE PLANNING/MANAGEMENT	STATE	MUNICIPAL	INDUSTRY
POTW PERMITS	In some cases EPA issues POTW Permits.	Review existing draft permits--incorporate permit specifications into WQM plan. Management phase--once plan has been adopted POTW Permit and PT standards must be compatible with plan	In some cases state issues POTW Permits.	Municipal POTW must meet requirements of NPDES Permit. Local PT Standards (listed in ordinance) should relate to NPDES permit requirements.	Compliance with PT Standards is required.
AREAWIDE PLAN	Reviews and approves Plan.	Establishes plan which may include specific: minimum pretreatment requirements; industrial waste survey procedures; ordinance types.	Certifies plan.	Contributes to WQM plan. In order to be eligible for federal funding must be consistent with 208 plans PT requirements.	Industry participates on advisory committee.
PLAN IMPLEMENTATION	Establishes financing provisions, approves designated management agencies.	Requires a management agency to run PT program.	State designates management agency(s). Develop State legislation such as enabling laws.	May have to amend ordinance to be in compliance with PT provisions of Areawide plan.	Committee participates on technical and policy committees.
PUBLIC PARTICIPATION	Holds public hearings on Federal PT Standards.	Arranges public hearings and information transfer sessions on relevant pretreatment issues. Assists municipalities in arranging public hearings on sewer use regulations.	Comment and review on pretreatment issues as they affect water quality standards and WQM plans.	Holds or participates in public hearings on new sewer use regulations. Provide forum for all interested persons.	Participates in public hearings on sewer use regulations.
MUNICIPAL ORDINANCE	Supports efforts such as the preparation of guidance documents.	Gives direct aid to municipality in ordinance development. Reviews all municipal ordinances within planning area. May recommend regional ordinance.	Aids municipality in ordinance development. Reviews ordinance for completeness.	Creates ordinance. Will be directly involved in implementation of ordinance.	Complies with ordinance requirements.
INDUSTRY WASTE SURVEY	Supports efforts such as the preparation of guidance documents.	In many cases will carry out survey. Identify discharges of toxic wastes.	May require municipality to modify survey approach.	Frequently carries out survey.	Provides information to satisfy survey requirements.

PT = Pretreatment
POTW = Publicly Owned Treatment Works

Although the NPDES does not require issuance of permits to industries discharging to sewers, the POTW that treats the waste is required to have a discharge permit. As part of the NPDES permit for the POTW, the municipal operating authority must submit forms identifying the discharges to the sewers from major contributing industries. (Figure 4.)

Pollutants that Interfere with POTW

Interference can be caused by a variety of chemical, biological, and physical phenomena. Interference consists not only of materials which inhibit biological sewage treatment processes, but also substances which cause problems in sewage collection systems, sludge disposal or utilization methods, water reuse, land application of wastewater or other operations. Interference with sludge disposal or utilization and reuse of wastewater consists primarily of incompatible pollutants which are concentrated in sludge or by reuse techniques.

Consideration should be given to interference with physical-chemical sewage treatment systems, employing activated carbon adsorption following primary treatment such as chemical coagulation and filtration. Although many facilities of this type are currently being designed and constructed, operating data from full scale physical-chemical plants is limited. The characteristics of the activated carbon adsorption process are such that few substances cause interference.

Materials Which Inhibit Biological Treatment Processes

Three basic categories of treatment processes have been delineated, including aerobic processes, anaerobic processes, and nitrification. The aerobic processes have been subdivided into activated sludge and trickling filter operations, while anaerobic processes consist of sludge digestion. Although many currently operating biological treatment plants utilize the trickling filter process, relatively little data is available on pollutant interferences. Information on aerobic biological treatment presented is predominantly concerned with the activated sludge process.

The general categories established for interfering substances are inorganics and organics, with acidity, alkalinity, pH, ammonia, transition metals, metals, sulfate and sulfide comprising the major components of the inorganic category. Organic substances include alcohols, amines, chlorinated hydrocarbons, pesticides and herbicides, phenol, surfactants and miscellaneous organic chemicals.

A precipitated pollutant has relatively little inhibitory impact compared to soluble components. Settleable solids are usually removed in primary treatment, and therefore, seldom reach the potentially inhibited biological unit process. Because the insoluble fraction of a pollutant parameter will have relatively little effect on biological treatment processes, most investigators have conducted their tests using only dissolved pollutants.

If a substance is inhibitory depends on factors, i.e., concentration and the presence of other chemicals which have synergistic or antagonistic effects. Some substances, such as mercury, present in wastewater at a very low concentration, can disrupt one or more functions of a biological treatment system. Others such as chloride ion, are inhibitory only at relatively high concentrations. Special phenomena that may alter the inhibitory nature of a substance are outlined below.

Synergism

Synergism is characterized as an increase in the inhibitory effect of one substance by the presence of another. Symergism, and its opposite antagonism, is most prevalent when considering combinations of transition metals or heavy metals. The inhibitory effects of these metallic constituents are also enhanced by acidity.

The synergistic effects of metals with acidity is understandable in terms of the chemistry of these metals. The transition and heavy metals tend to be insoluble by hydrolysis in the pH range of sewage influent. They precipitate or absorb on solids, and interact with polyelectrolytes or various chemical species

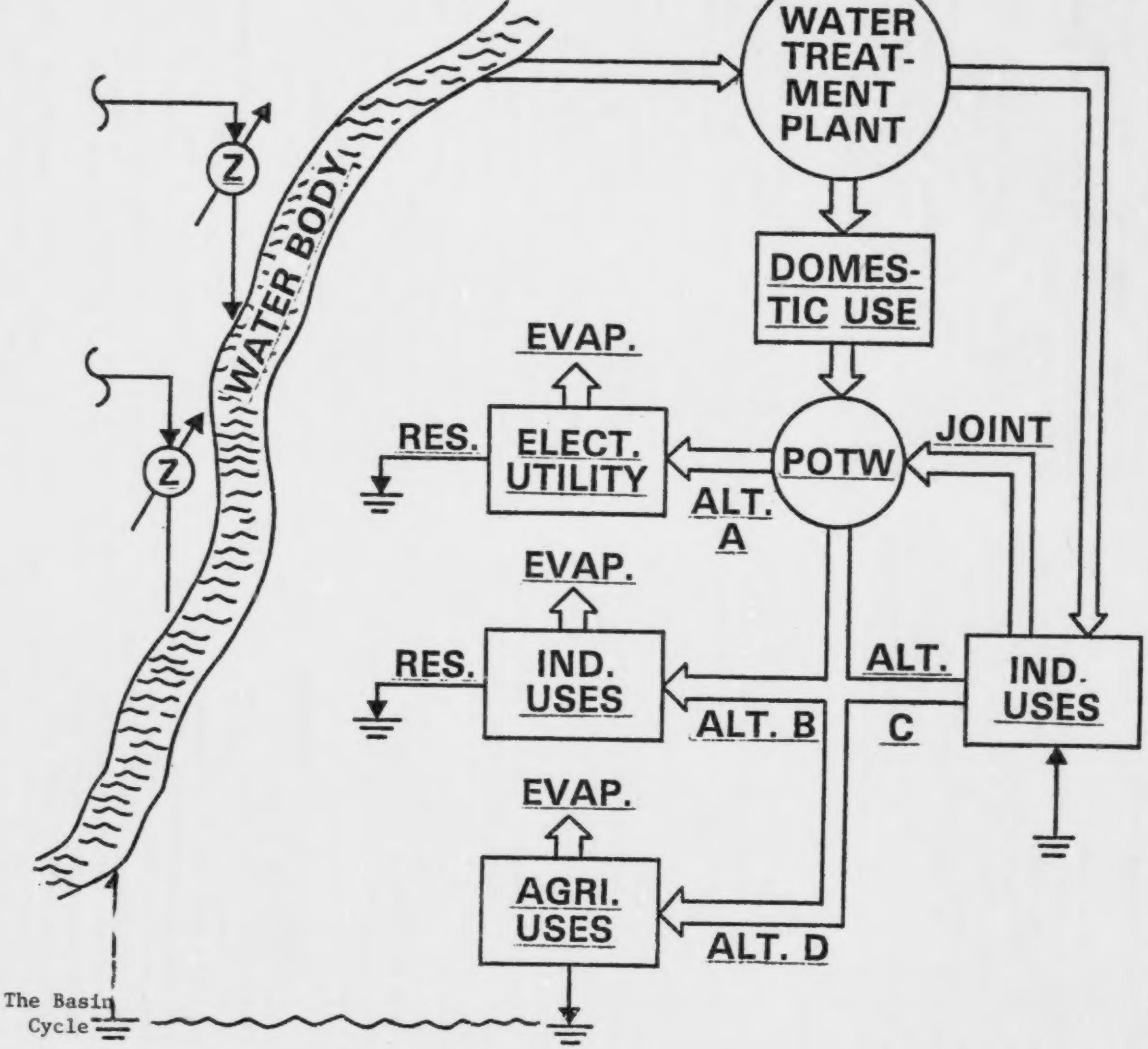


Figure 4. The Basin Cycle

containing anionic functional groups. Acidity suppresses hydrolysis, and the hydrogen ion competes with the metal ion for adsorption sites on solids or anionic functional groups in solution.

The synergism of cyanide or other complexing substances makes metal wastes more easily biodegradable. It is possible for the microorganism to ingest excessive levels of complexed metal ion and then to destroy by assimilation the complexing substance shielding the microorganim from the metal ion with the release of an excessive level of the metal within the organism, upsetting its biological life processes.

Antagonism

Antagonism is the opposite of synergism; it is characterized as a decrease in the inhibitory effect of one substance by the presence of another. Antagonistic effects occur with the combination of metallic and anionic pollutants. Chelating agents, such as EDTA (Disodium salt of ethylenediamine tera-acetic acid) and HEDTA (disodium salt of hydroxy-ethylenediamene triacetic acid) exhibit antagonistic properties with metals. Chelating agents are used in culturing microorganisms to regulate the level of metals needed to grow bacterial cultures. Bacteria survive in culture solutions containing concentrations of these metals high into the inhibitory range, when chelating agents are present.

Ambiguity in the inhibitory effects of metals on sludge digestion exist in the literature. The ambiguity may be explained by the antagonistic effect of the sulfide normally present in a digester. Sulfide ion precipitates metals, removing them from solution and, therefore, eliminating their inhibitory effect. This has been used effectively in restoring upset digesters to operation. Additionally, sulfide, other ions such as hydroxide chromate, ferro-cyanide, phosphate, carbonate, and arsenate will tend to precipitate with metals, thereby reducing the inhibitory effect.

Acclimation

Significant biological factors must be considered to understand the inhibitory effect of pollutants. In the activated sludge process, a healthy biomass contains a broad distribution of microorganisms, including many species of bacteria and protozoa. In the absence of sufficient food, reproduction slows and the microorganisms devour each other. Whenever the environment changes due to the introduction of, or omission of a given pollutant, the opportunities for reproduction and growth of different species change.

The history of an activated sludge biomass affects how it will respond to a new pollutant. When a new pollutant is introduced, species which cannot tolerate this substance fail to reproduce and grow, and tend to die off, while more tolerant species consume the food supply and grow and reproduce.

When a biomass becomes accustomed to the presence of a normally inhibitory concentration of a substance, it can be characterized as acclimated to that pollutant. Sludge digestion and nitrification do not have the same flexibility of adaptation to changing environmental conditions as do other biological processes. Both nitrification and sludge digestion are biological processes that rely on particular strains of bacteria. Sludge digestion process proceeds in two steps, using two specific bacterial strains to achieve digestion. Nitrification is also limited to particular bacterial types. When adverse conditions are encountered in these processes, there is no possibility of another organism taking over for the affected strain of bacteria. Neither nitrification nor sludge digestion are readily acclimated to a new pollutant and may be easily upset.

A POTW may contain any of thousands of inorganic compounds present as major or minor constituents. Most inorganic substances dissolved in wastewater are present in ionic form, and can reduce the number of parameters of interest to a few cations and anions and non-ionics.

The major cations consist of the ammonium ion and various metal ions. A few metal ions: i.e., sodium, calcium, and magnesium, are prevalent in wastewaters, but are not inhibitory except at very high concentrations (a few mg/l, order of magnitude) they become inhibitory to biological processes.

Discharges of wastes from manufacturing or processing industries may result in wastes pH from about 3 to 11.

Acidic contributions are common from many industrial processes including metal cleaning, plating or treatment plants, manufacture of drugs and explosives, processing of photographic films. etc.

Alkaline contributions are from laundries, detergent manufacture, bottle cleaners (dairies, for example), textile processing and cement manufacturing.

Summary of Inhibitory Properties:

1. The activated sludge process can treat waste solutions ranging from about pH 3 to 11, provided that the pH level has been stabilized.

2. Inhibitory effects are related to temperature and were noted above pH 10 at 20°C and above pH 9 at 10°C.

3. At pH 4 the activated sludge process is 43% effective.

4. The sludge digestion process is extremely sensitive to pH, and deviations of pH outside the range 6.5 to 7.5 generally cause upset of sludge digesters.

TABLE 2

THRESHOLD CONCENTRATIONS OF INORGANIC POLLUTANTS THAT ARE INHIBITORY TO BIOLOGICAL TREATMENT PROCESSES

POLLUTANT	CONCENTRATION (mg/l)			REFERENCES
	ACTIVATED SLUDGE PROCESSES	ANAEROBIC DIGESTION PROCESSES	NITRIFICATION PROCESS	
Ammonia	480	1500		9,10,13
Arsenic	0.1	1.6		2,11
Borate(Boron)	0.05-100	2		2,6,23
Cadmium	10-100	0.02		11,13,18
Calcium	2500			
Chromium (Hexavalent)	1-10	50	0.25	2,13,14
Chromium (Trivalent)	50	50-500		15
Copper	1.0	1.0-10	0.005-0.5	2,12,14,21,24
Cyanide	0.1-5	4	0.34	2,11
Iron	1000	5		11
Lead	0.1		0.5	2
Manganese	10			11
Magnesium		1000	50	
Mercury	0.1-5.0	1365		11
Nickel	1.0-2.5		0.25	24
Silver	5			6,11
Sodium		3500		8.9
Sulfate			500	8,9
Sulfide		50		
Zinc	0.08-10	5-20	0.08-05	1,2,3,4

Note: Concentrations shown represent influent to the unit processes

TABLE 3

THRESHOLD CONCENTRATIONS OF ORGANIC POLLUTANTS THAT ARE INHIBITORY TO BIOLOGICAL TREATMENT PROCESSES

POLLUTANT	CONCENTRATION (mg/1)			REFERENCES
	ACTIVATED SLUDGE PROCESSES	ANAEROBIC DIGESTION PROCESSES	NITRIFICATION PROCESSES	
Alcohols				
Allyl		100	19.5	8.24
Crotonyl		500		8
Heptyl		500		8
Hexyl		1000		8
Octyl		200		8
Propargyl		500		8
Phenols				
Phenol	200		4-10	16
Creosol			4-16	16,24
2-4 Dinitrophenol			150	24
Chlorinated Hydrocarbons				
Chloroform		10-16		8,22
Carbon Tetrachloride		10-20		17,21,19
Methylene Chloride		100-500		19
1-2 Dichloroethane		1		21
Dichlorophen*		1		21
Hexachlorocyclohexane		48		21
Pentachlorophenol*		0.4		21
Tetrachloroethylene		20		21
1,1,1,-Trichloroethane		1		21
Trichloroethylene		20		21
Trichlorofluoromethane*		0.7		21
Trichlorotrifluoroethane (Freon)		5		21
Allyl Chloride			180	24
Dichlorophen			50	24
Organic Nitrogen Compounds				
Acrylonitrile		5		21
Thiourea			0.075	21,24
Thioacetamid			0.14	21,24
Analine			0.65	24
Trinitrotoluene (TNT)	20-25			
EDTA	25		300	24
Pyridine			100	16

TABLE 3 (Continued)

POLLUTANT	CONCENTRATION (mg/1) ACTIVATED SLUDGE PROCESSES	ANAEROBIC DIGESTION PROCESSES	NITRIFICATION PROCESSES	REFERENCES REFERENCES
Surfactants				
Nacconol	200			
Ceepryn	100			
Miscellaeous Organic Compounds				
Benzidine	500	5		7
Thiosemicarbazide			0.18	24
Methyl isothiocyanate			0.8	24
Allyl isothiocyanate			1.9	24
Dithio-oxamide			1.1	24
Potassium thiocyanate			300	24
Sodium methyl dithiocarbamate			0.9	24
Sodium dimethyl dithiocarbmate			13.6	24
Dimethyl ammonium dimethyl dithiocarbamate			19.3	24
Sodium cyclopentamethylene dithiocarbamate			23	24
Piperidinium cyclopentamethylene dithiocarbamate			57	24
Methyl thiuronium sulphate			6.5	24
Benzyl thiuronium chloride			49	24
Tetramethyl thiuram momosulphide			50	24
Tetramethyl thiuram disulphide			30	24
Diallyl Ether			100	24
Dimethyl-paranitrosoaniline			7.7	24
Guanidine carbonate			19	24
Skatole			16.5	
Strychnine hydrochloride			175	24
2 chloro-6 trichloromethyl-pyridine			100	24
Ethyl urethane			250	24
Hydrazine			58	24
Methylene blue			100	24
Carbon disulphide			35	24
Acetone			840	24
8-hydroxyquinoline			73	24
Streptomycin			400	24

Note: Concentrations shown represent influent to the unit process. Where indicated with a *, the concentration represents total plant influent.

THE ENVIRONMENTAL PROTECTION AGENCY RESEARCH AND DEVELOPMENT JOINT INDUSTRIAL-MUNICIPAL POINT SOURCES

The "Joint" program area is defined as the technology research oriented to supplying POTW's with the technology needed for meeting effluent standards for mixed industrial, mixed industrial and municipal, and water supply point sources of discharge. The qualifying conditions for relevancy to these point sources include the involvement or sponsorship of a public treatment authority and the condition that the industrial portion of the wastewater be the controlling wastewater management factor.

In terms of point source standards impact, the joint systems have direct bearing on the following standards and related technology achievement levels: (1) Toxic Standards, (2) Industrial BPT, BAT, and NSPS (New Source Performance Standards), and (3) POTW's Standards for Pretreatment, Secondary, and BPWTT (Best Practical Wastewater Treatment Technology).

The wastewater characteristics and related complexities of control vary as the type of industry input changes.

The technology for 1977 implementation is best referenced to the 30 Phase I guidelines for industrial and municipal sources mentioned earlier. Generally speaking, biodegradable organics and related suspended solids are controllable using bio-oxidation and clarification processes. The availability of viable controls for incompatible pollutants such as inorganic, toxic and residual-refractory pollutants varies from undemonstrated to inadequate effectiveness.

The prime objective of the R&D program is to demonstrate adequate, practicable and economically achievable technology for the elimination of the discharge of pollutants (closed cycle) for 3 of the 4 problem areas--Area Wide, Joint, Residual and Refractory. Secondary and interim milestones also exist in the total environmental and open cycle waste management alternatives. The pretreatment segment of the program is concerned with improving the compatibility of industrial wastewaters for discharge to municipal systems.

The area wide treatment and control problem area includes those situations in which wastewaters are collected, combined and treated for a broad geographical area. This subcategory includes: wastes from several industries combined with domestic wastes from an industrial complex, and wastes from a metro-area, or even from an entire river basin.

The joint industrial/municipal problem area includes those situations where only one type of industry discharges their wastes into a municipal treatment plant.

The residual and refractory problem area includes those industrial and municipal wastes which are in concentrated form and are either toxic or incompatible with existing biological treatment systems. Because of these factors, the wastes can be most effectively managed by a common collection and treatment system which must achieve BAT or Closed Cycle effectiveness. These wastes include: liquid concentrated (paints, pigment dyes, solvents, oils, acids, alkalies), sludges (inorganic salts, organic metallic oxides), and water treatment sludges. By providing alternate disposal techniques for these materials, they can be removed from municipal systems and landfills where they receive inadequate stabilization.

The pretreatment problem area is concerned with the treatment given an industrial wastewater prior to its discharge to a municipal sewer. Technology research will be carried out to evaluate the economic and technology impact of pretreatment guidelines at the industrial-municipal interface and to define the criteria for industrial compatibility and incompatibility relative to municipal treatment practice.

Copies of reports on EPA's Joint Industrial/Municipal demonstration projects are available.

The EPA schedule for Promulgation of Pretreatment Standards (NRDC vs. Train Consent Decree) is shown on TABLE 4.

TABLE 4. EPA SCHEDULE FOR PROMULGATION OF PRETREATMENT STANDARDS
(NRDC VS. TRAIN CONSENT DECREE)

POINT SOURCE CATEGORY	PROMULGATION DATE
TIMBER PRODUCTS PROCESSING	OCTOBER 15, 1976
NONFERROUS METALS MANUFACTURING	OCTOBER 15, 1977
LEATHER TANNING & FINISHING	FEBRUARY 15, 1977
PETROLEUM REFINING	FEBRUARY 15, 1977
STEAM ELECTRIC POWER PLANTS	FEBRUARY 15, 1977
ELECTROPLATING	MAY 15, 1977
TEXTILE MILLS	MAY 15, 1977
INORGANIC CHEMICALS MANUFACTURING	MAY 15, 1977

PRETREATMENT STANDARDS FOR THE FOLLOWING POINT SOURCE CATEGORIES WILL BE PROMULGATED BY DECEMBER 31, 1979

IRON AND STEEL MANUFACTURING
PAVING AND ROOFING MATERIALS (TARS AND ASPHALT)
PAINT AND INK FORMULATION AND PRINTING
SOAP AND DETERGENT MANUFACTURING
PLASTIC AND SYNTHETIC MATERIALS MANUFACTURING
PULP AND PAPERBOARD MILLS; AND CONVERTED PAPER PRODUCTS
RUBBER PROCESSING
MISCELLANEOUS CHEMICALS
MACHINERY AND MECHANICAL PRODUCTS MANUFACTURING
ORE MINING AND DRESSING
COAL MINING
AUTO AND OTHER LAUNDRIES
ORGANIC CHEMICALS

Conclusions

Based upon more than 18 separate demonstration projects dealing with joint industrial-municipal treatment covering all major industrial effluents, funded by EPA's Office of Research and Development, the following conclusions can be drawn:

1. In most cases there are real capital, operational, and treatment advantages to joint municipal-industrial.

2. The institutional relationships required by either joint treatment and pretreatment usually results in better planning, management, cooperation, financing, and ultimately higher quality effluents and a cleaner environment.

   Actual full-scale field experiences have repeatedly demonstrated to us in EPA's Office of Research and Development that areawide analysis, evaluation, and planning results in more cost effective projects.

3. It is important to monitor industrial wastewater discharges to protect the collection and treatment system.

4. Pretreatment by industry to reduce certain high levels of SS, BOD, toxic and harmful ingredients to acceptable levels is normally cost effective.

5. Increased hydrolic flows can reduce peak to average flows.

6. Utilization of the nutrients available in domestic wastes for biotreatment of industrial wastes is advantageous.

7. Joint treatment can result in sludge disposal or utilization problems.

8. Industries discharge significant concentrations of metals, toxics, cyanides, and complexing agents, because dilution in the overall wastewater flow generally lowers the concentrations of these materials to tolerable levels that normally will not adversely affect biological treatment. Nevertheless, the potential for such interference exists.

Future

High on the Environmental Protection Agency's priority list for the coming year is the selection of a national pretreatment strategy for handling industrial effluent discharges into POTW. EPA will soon propose four options for public consideration and comment:

1. A combination of national technology-based standards for as many as 21 industries, with voluntary local enforcement programs.

2. Federal standards for only the most hazardous pollutants, which would be applied only to the most significant plants discharging those pollutants. Other pollutants would be controlled by local programs.

3. A variance that would allow publicly-owned treatment works to develop and enforce locally-derived pretreatment limits in lieu of Federal standards whenever water quality standards are being met.

4. Direct Federal-State envorcement of Federally-developed national standards. This option would involve technology-based standards for all incompatible pollutants within the next three years.

The USA effort to clean the waterways has made tremendous progress in the past few years. More than 90 percent of all municipalities and industries discharging into our lakes and streams are either complying with pollution control standards or are on definite water clean-up schedules.

# # #

FINAL REPORTS AVAILABLE
JOINT INDUSTRIAL/MUNICIPAL POINT SOURCES

11060 EOC 07/69	Joint Municipal and Semichemical Pulping Wastes, City of Erie, Pennsylvania and Hammermill Paper Co. GPO-$1.50
11060 FAE 11/69	Feasibility of Joint Treatment in a Lake Watershed, Onondage County, Syracuse, N. Y. GPO-$1.50
11060 FAE 04/71	Onondaga Lake Study,, Onondaga County, Syracuse, N. Y., GPO-$4.50
11060 DPD 02/71	Combined Treatment of Municipal Kraft Linerboard and Fiberboard Manufacturing Wastes; by Board of Water Commissioners, City of Macon, Georgia, Georgia Kraft Co., and Armstrong Cork Co. GPO-$1.25
12130 EDX 07/70	Joint Treatment of Municipal Sewage and Pulp Mill Effluents, the Green Bay Metro Sewage District, Green Bay Wisconsin. GPO-$6.00
12130 EZR 05/71	Combined Treatment of Domestic and Industrial Wastes by Activated Sludge, City of Dallas, Oregon. GPO-$1.25
12130 EGK 06/71	Biological Treatment of Chloriphenolic Wastes, City of Jacksonville, Arkansas. GPO-$1.50
12130 FQJ 06/71	Pollution Abatement and By-product Recovery in Shellfish and Fisheries Processing, Food Chemical and Research Labs., Inc., Seattle, Washington, and Engineering Science of Alaska, Anchorage, Alaska. GPO-$1.00
12130 DUJ 09/71	Whey Effluent Packed Tower Trickling Filtration, by Quirk, Lawler & Matusky Engineers, Tappan, New York. GPO-1.50
EPA 660 2-73-038 3/74	Deepwater Pilot Plant Feasibility Study, Delaware River Basin Commission, Trenton, New Jersey GPO-$4.10
12120 FYF 03/72	Fluidized-Bed Incineration of Selected Carbonaceous Insustrial Wastes, State of Ohio, Dept. of Natural Resources, Columbus, Ohio GPO-$1.00
12010 DTQ 02/72	Combined Steel Mill and Municipal Wastewater Treatment, Weirton Steel Division, National Steel Corp., Weirton, West Va. GPO-$1.50
EPA 660 2-73-010 12/73	Treatment of Domestic Wastewater and NSSC Pulp and Paper Mill Wastes, Harriman Utility Board, Harriman, N. Y. GPO-$1.40
EPA R2-73-236 11060 DLF	Tertiary Treatment of Combined Domestic and Industrial Wastes, by J. W. Lee, CH2M/HILL, Corvallis, Oregon. GPO-#2.60
EPA 660/2 74020	Evaluation of Polymetric Clarification of Meat-Packing and Domestic Wastewater, by K. D. Larsen and D. A. Maulwauf Metropolitan Sewer Board, South St. Paul, Minnesota GPO

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6. *Environmental Effect of Photoprocessing Chemicals*, Vol. II, Report by the National Association of Photographic Manufacturers, Inc., 600 Mamaroneck Ave., Harrison, N. Y. 10528, 324 pp. (1974)

7. *Fate of Benzidine in the Aquatic Environment: A Scoping Study*, U.S. EPA Contract # 68-01-2226, January, 1974.

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14. "Effects of Metallic Ions on Biological Waste Treatment Processes," Reid, George W. et.al., *Water and Sewage Works*, Vol. 115, No. 7, p. 320, (July 1968)

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GLOSSARY

Acclimatization (Acclimation) - A process by which the biological organisms in a biological treatment system develop the capability of maintaining normal life cycles in the presence of concentrations of pollutants that would normally inhibit their activity.

Antagonism - Reduction of the inhibitory effect of one substance by the presence of another.

Compatible Pollutant - A pollutant which is normally treated by conventional biological sewage treatment processes.

Contribution (Contributors) - A point source originating within the area served by a sewerage system and treatment works.

Existing Source - Any wastewater source, which was contributing to a sewerage system (or a potential source that was under construction), at the time of publication of proposed regulations.

Federal Water Pollution Control Act Amendments of 1972 - Public Law 92-500 which provides the legal authority for current EPA water pollution Abatement projects, regulations and policies.

Incompatible Pollutant - Any pollutant which is not normally treated by conventional biological sweage treatment processes.

Industrial Waste Ordinance - An enforceable local law which establishes rules, regulations, limitations and prohibitions to be adhered to by wastewater sources within the service area of a publicly owned sewer system.

Inhibition - A contribution that decreases the pollution removal efficiency of a biological treatment facility.

Interference - A contribution that hinders in any way the operation of a wastewater collection and treatment system. Inhibition can be considered a type of interference.

Joint Treatment - The treatment of a wastewater consisting of a combination of domestic and industrial flows in a single treatment system which has been specifically designed to handle the combined wastewater.

Limited Waste - A waste, which by either local ordinance or Federal regulation, cannot be present in a contribution or direct discharge above a certain concentration.

Major Contributing Industry (MCI) - A major source of industrial wastewater within a POTW system, as defined in the Federal pretreatment standards.

Monitoring - The practice of investigating, surveying and/or sampling wastewater sources in an effort to obtain information on the quality or quantity of the wastewater flow.

National Pollutant Discharge Elimination System (NPDES) - A system of permits to discharge wastewaters to navigable waters developed under the authority of Section 402 of Public Law 92-500. The permit system has as its objectives the achievements of the goal of that law, the elimination of the discharge of pollutants by 1985.

New Source - A wastewater source for which construction began after the publication of proposed regulations.

Pass Through - The discharge to the receiving waters from a publicly owned treatment works of a pollutant without substantial modification or removal.

Pretreatment - The Treatment of a wastewater contribution, at the point of or prior to release to a publicly owned treatment system.

Prohibited Waste - A material which must be excluded from any discharge of wastewater to a collection system in any concentration. Prohibited wastes include, among others, explosive or corrosive materials.

Publicly Owned Treatment Works (POTW) - A sewerage facility which collects, treats or otherwise disposes of wastewaters, owned and operated by a village, town, county, authority or other public agency.

Synergism - An increase in the inhibitory effect of one substance caused by the presence of another.

Upset - A substantial decrease in the treatment efficiency of a biological treatment system caused by changes in the life cycles of the biological organisms comprising the system.

CRITERIA AND ASSESSMENT OF WASTE TREATABILITY

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ABSTRACT

In the past, wastewater treatment plants were designed to deal primarily with domestic sewage. Many industrial organic wastes do not have the same degree of amenability to treatment. The frequency with which difficulties appear at municipal wastewater treatment plants has increased as industrial effluents have become a larger proportion of the sewage. Many industrial effluents are not readily amenable to biological treatment and specially designed processes will have to be used to deal with the more biologically resistant materials. For unknown wastes, the degree of treatability must be established before adequate treatment facilities can be designed. This paper discusses several methods that can be used to determine the treatability of compounds and industrial wastes. The important features and limitations of each test method are presented.

INTRODUCTION

Traditionally, nature provided the ultimate disposal of man-made waste, but as man advanced technologically, nature's work became more difficult. In fact, for some of the newer man-made substances, nature fails to degrade them. Three examples that readily come to mind are plastic packaging material, some of the detergents, and pesticides. Thus, the accumulation in the environment of many of the newer products synthesized by man has caused considerable interest to be focused on biological degradation and treatability of wastes.

Each year hundreds of new products are developed by the expanding chemical and petroleum industries. Many of these reach our surface waters. In addition, each synthesis step results in the formation of many side-products and by-products. A portion of these and a low concentration of the principal product end up in the wastes from the process. The industrial waste producer must determine: (1) If the particular waste is amenable to biological treatment or is it inhibiting or toxic to microbial activity? (2) If the nature and quantity is acceptable at the local wastewater treatment plant? (3) What design parameters of a conventional biological wastewater treatment system must be altered to treat the wastes? (4) If the waste is to be treated on site, what treatment is most suitable? This paper discusses several methods which can provide answers to these questions.

For unknown wastes, the degree of amenability must be established before adequate treatment plant designs can be developed. Properly designed processes can make some undesirable wastes more amenable to biological treatment. Some wastes will cost less to treat if done separately at their origin while they are in a concentrated form. The difficult cases to decide are the ones where wastes are not completely inhibitory to the treatment process. In this case, they interfere with the treatment process to the extent that the effluent quality is lowered and the capacity of the plant must be increased to accommodate them. Some industrial effluents are not readily amenable to biological treatment and specially designed processes will have to be used to deal with the more biologically-resistant materials.

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The treatment processes of the future will no doubt be a combination of biological and physical-chemical processes.

A further aspect of the problem of treatability occurs with sludge digestion. Any toxic material which separates with the sludge might affect the sludge digestion process. There appears to be a growing awareness in the United States of digester troubles without an equal cognizance of the causes of the troubles. The effect of very low concentration of chlorinated organic compounds on methane production by a digester is remarkable.

Through the years there has been a shift in public policy in the United States concerning water pollution. At first there was a concern for eliminating obvious nuisances, then a shift to concern for public health, then to ecological balance and finally today for preserving the natural beauty of the water. The 1972 Federal Water Pollution Control Act Amendments incorporated all these concerns. As a national goal, it calls for zero pollution discharge into American waters by 1985, and the achievement of water quality safe for all marine and freshwater life and human recreation by 1983. This law has put more emphasis on the treatability of wastes. This Act prevents the discharge of a pollutant to a publicly owned treatment works which interferes with or passes through or otherwise is incompatible with such works. The Toxic Substances Control Act which was enacted on October 11, 1976, poses particular challenges to the industries manufacturing toxic substances. This law authorizes the U.S. Environmental Protection Agency to obtain test data from industry and to prohibit or limit the manufacture and distribution in commerce of chemical substances which present an unreasonable risk to health or the environment. Manufacturers of new chemical substances must notify the Administrator of EPA at least 90 days before the manufacture of the chemicals for commercial purposes. Chemicals used exclusively in pesticides, food, food additives, drugs and cosmetics are exempted from the Act. These chemicals are covered by other regulations. Both of these laws indicate that serious consideration must be given to the treatability of new products if they will become a constituent of wastewater.

BIODEGRADABILITY

Concepts and Criteria

Biodegradability has now become a well-known word by everyone who has become interested in the pollution problem. To many it means halting the spiraling mounds of garbage or the development of a plastic bottle that will disintegrate. For some of the newer man-made substances nature fails to degrade them. Thus, the accumulation in the environment of many of the newer products synthesized by man has caused considerable interest to be focused on biological degradation and treatability.

Biodegradable has been defined in several ways. It can be defined as the removal of organic matter from solution by enzymatic processes of oxidation and assimilation. Assimilation is the conversion of organic substances or substrates to new bacterial protoplasm. Most certainly, the word degrade means to reduce the complexity of the substances. If we define biodegradation in terms of energy, then biodegradation consists of the consumption of a substance by biological action of living organisms with an accompanying degradation of energy. A layman's definition would be an organic substance plus microorganisms gives carbon dioxide, water and more organisms.

It should be noted that in none of these definitions were the factors of time and extent mentioned. In the real world of wastewater treatment, the extent of biodegradation in a fixed time limit is a very important consideration in any working definition of biodegradability. Some researchers prefer not to use the term biodegradability in connection with wastewater treatment processes. They prefer to use the term treatability in describing whether a substance is removed by biological waste treatment processes. The thinking is that in dealing with waste treatment processes many constraints have been placed on the microorganisms that might prevent a compound from degrading. This argument is mostly from an academic point of view rather than from a practical point of view. Why are we mainly concerned about biodegradability? Are not we really concerned about the nuisance value of a new substance and whether it is eliminated by wastewater treatment processes if it finds its way into the wastewater? In principle, laboratory testing should show the true biodegradability of the material tested as it would occur in the environment or in the waste treatment plants.

Obviously, most substances will ultimately be stabilized by microbial action, given enough time and the right conditions We must then consider the biodegradation process as a rate process. For practical purposes of wastewater treatment, the substances should degrade at a rate equal to or higher than the normal constituents of domestic waste after the bacteria have become acclimatized to the substances. In the activated sludge process, the time limit is four to eight hours. In this definition we are comparing one substance with another. The biodegradability of a material cannot be expressed as a single absolute number, but as a relative number because of the complexity of the biological degradation process. It is well that attention is directed to the fact that biodegradability is not a property or condition of the water that can be measured, but rather a property of the pollutant.

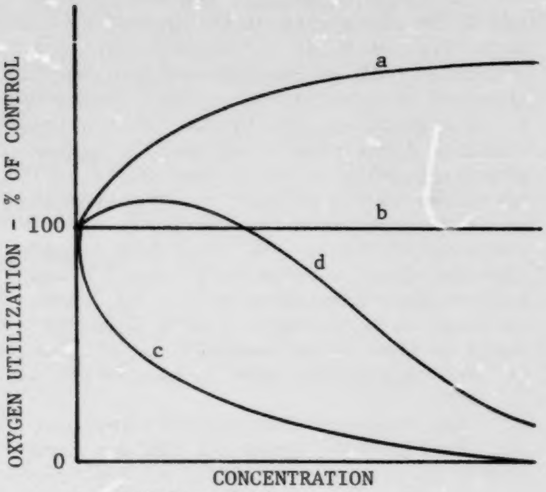
For any compounds to be degraded, the proper conditions must be present. History bears witness to materials that have not been degraded in thousands of years which are known to be degradable under appropriate conditions. For example, pieces of hair, logs, grain, rope, etc., have been found. Microorganisms cannot grow, and hence, organic compounds cannot be degraded unless a number of conditions are fulfilled. All necessary nutrients and growth factors must be present in the appropriate chemical and physical form. Matter in solution is usually more readily assimilated than that in suspension. Finely divided suspensions are more readily attacked than coarse suspensions. An adequate supply of water is necessary and sufficient supply of hydrogen acceptors, such as molecular oxygen, nitrate or carbon dioxide. There are optimal values of temperature and pH value, but growth will occur over wide ranges. It has been claimed that microorganisms exist that will ultimately degrade recalcitrant substances, given enough time. This has little practical significance if the rate of discharge to the environment exceeds the rate of breakdown. Biodegradation may continue beyond the treatment in surface waters, but its rate will be slower particularly in the sea.

Although we usually think a compound has to be degradable to be removed by wastewater treatment, it is possible that the compound could be removed entirely by flocculation or adsorption. Toilet tissue may be considered as falling into this category. Although it is slowly degraded by microorganisms, most of it is removed by sedimentation in the primary sedimentation tank [illegible]y adsorption on the bacterial floc in the aeration tank. The activated sludge floc has great adsorption properties and many organic substances are so adsorbed. It is true, in most cases, that a compound would be more environmentally acceptable if it were rapidly degraded by microorganisms.

How biodegradable should a compound be? The possibilities range all the way from slight to ultimate breakdown of the compound to cell matter, carbon dioxide, and water. Biodegradability must be measured in terms of its effect on the environment. In widespread use of a specific compound, a potential exists for significant environmental pollution if the compound is not degraded within a reasonable time in wastewater treatment plants or rivers. The future would indicate that environmentally acceptable biodegradability for products, which become waterborne wastes, should be that they are completely degraded to products that are harmless in surface and drinking waters.

Toxicity to Biological Systems

The two most important questions about biological wastewater treatment processes related to toxic wastes are: (1) the sensitivity of the biological processes to the effects of poisons, (2) the efficacy of the treatment process in removing the toxic substances. The sensitivity of biological treatment processes to toxic wastes is reduced by several factors. The microorganisms which are beneath the surface of the biological film in trickling filters and in the center of the activated-sludge flocs are protected to some extent from shock discharges. Bacterial populations have the remarkable ability to acclimatize to many toxic compounds and utilize them as substrates. Finally, even if the population of microorganisms is severely depleted by toxic substances poisoning, it can rapidly recover, because the generation time is quite short. Figure 1 illustrates different types of oxygen utilization. Curve C is for a toxic waste.



a. Biodegradable
b. Non-degradable
c. Toxic
d. Toxic at high concentrations

Figure 1. Common oxygen utilization curves.

Curve D is for a waste that is biodegradable at low concentrations but becomes toxic as the concentration increases.

Anaerobic digestion is the most susceptible of the biological treatment processes to poisoning by toxic substances, especially to heavy metals and chlorinated organic compounds. Anaerobic digestion is two biological processes rolled into one. The first group of bacteria degrade the starches, fats, and proteins to mostly short-chain fatty acids. The second group (methane bacteria) is much more susceptible to inhibition than the first. Reduction of methane production signals digester failure which results in offensive sludge that is not well-digested. The effect of chlorinated organic compounds at very low concentrations is remarkable, and constitutes a warning that close watch must be kept on the disposal of new chlorinated hydrocarbons which come on the market. Toxic levels of a number of organic substances are shown in Table 1.

The effects of metals on anaerobic digesters are dependent on the amount of sulfides present. Most digesters will contain enough sulfides so that failures are not usually due to metals except for slug dosages from spills, etc. Some of the more common metals and the acceptable concentration to digesters are shown in Table 2. Although the resilience of bacterial populations has been stressed, this is more true of aerobic processes than with anaerobic digestion. Once the performance of a digester has deteriorated, it is usually rather difficult to reestablish.

In addition to the ability of biological processes to acclimatize to oxidize certain toxic organic wastes, aerobic processes of wastewater treatment can acclimatize to a certain extent to the presence of heavy metal ions. However, in the case of metals, they are not oxidized but are absorbed or adsorbed by the activated sludge floc or trickling filter film. The same thing occurs with many biologically resistant organic compounds. They can be removed by biological treatment processes by adsorption without being oxidized. Many volatile compounds are stripped from the wastewater in the aeration tank of the activated sludge processes.

Nitrification is the aspect of aerobic biological treatment most sensitive to toxic materials. The autotrophic bacteria which oxidize the ammonia are much more readily inhibited than are the heterotrophic bacteria which are the ones that oxidize carbon compounds. This effect is shown in Table 3. The main impact of different classes of toxic substances is indicated in Table 4. The table is not meant to be all encompassing, but merely to indicate where the main impacts are in considering different classes of waste.

Measuring Biodegradability

As in any chemical reaction, the process of biodegradation can be followed by the observation of the disappearance of the reactants (organic compound and oxygen) or the appearance of products (cell matter, CO_2 and inorganic ions). The disappearance of some specific organic compounds cannot in many cases be proof that the compounds have completely degraded. Most chemical or physical tests depend on some functional group of the organic compounds. For example in testing for a surfactant of the alkyl benzene sulfonate type, several methods of assay could be used. If the method were based on the benzene ring, then the benzene would have to be degraded for it not to be detected. However, if the method were based on the

TABLE 1. TOXICITY TO ANAEROBIC DIGESTERS

Organic Compound	Concentration mg/l In Sewage	 In Sludge
Acrylonitrile[1*]		5
Carbon tetrachloride[2]		10
Chloroform[3]	0.1	
1,2 Dichlorethane[4]		<1
Methylene chloride[3]	1	
Di-(5-Chloro-2-hydroxyphenyl) methane (Dichlorophen)[5]	1	
Pentachlorophenol[2]	0.4	
1,1,1-Trichloroethane[2]		1
Trichlorofluoromethane[2]	0.7	

*Numbers denote references.

TABLE 2. TOXICITY OF METAL IONS TO ANAEROBIC DIGESTION

Metal Ion	Concentration in Sewage (mg/l)
Cadmium[2*]	1
Copper[6]	0.7 - 1
Dichromate (as Cr)[7]	10
Lead[8]	50 - 70
Nickel[9]	40
Tin[5] (as Sn)	9
Zinc[10]	10

*Numbers denote references.

formation of sulfate, then the molecule would have to be so disintegrated to have the sulfonates split off the compound.

For compounds or unknown wastes lacking specific analysis of sufficient sensitivity and applicability in biological system, a nonspecific analysis must be used. The most used methods have been the oxygen utilization or biochemical oxygen demand (BOD), the chemical oxygen demand (COD), and the total organic carbon (TOC). Unlike the methods which concern the observation of the disappearance of some specific organic compound, the disappearance of oxygen as the degradation proceeds is a general function of aerobic systems. It is a direct measure of biological oxidation, and it is reasonable to infer that biological oxidation has taken place if oxygen is consumed and carbon dioxide is produced in greater amounts than the control.

Of the three methods, only the carbon determination is a definitive measurement. The BOD and COD tests are empirical tests which give results in terms of the amount of oxygen that is utilized by matter under a prescribed analytical procedure. They do not indicate the quantity of any particular constituent; therefore, it is essential that they be conducted according to a standard method. A general discussion of the BOD, COD, and TOC methods follows.

TABLE 3. TOXICITY OF METAL IONS TO AEROBIC PROCESSES

Metal Ions	Concentration mg/l Oxidation of Organic cpds.	 Oxidation of ammonia
Cadmium[11,12]	1	5
Copper[7]	25	1
Copper[12]		2
Dichromate[7] (as Cr)	5	
Dichromate[13] (as Cr)	1	1
Dichromate[14] (as Cr)		10
Lead[15]	10	
Nickel[7]	2.5 - 10	
Nickel[16]		1
Nickel[12]		2
Zinc[7]	20	
Zinc[17]	25 - 55	
Zinc[18]	2	
Zinc[12]		1

*Numbers denote references.

TABLE 4. IMPACT OF TOXIC SUBSTANCES ON TREATMENT PROCESSES & WATERCOURSES

Class of Substances	River Life	Anaerobic Digestion	Aerobic Treatment
Metal ions	+	+	+
Chlorinated organic cpds.	+	+	
Organophoso cpds.	+		
Aromatic nitro cpds.	+		
Cyanides	+		+
Arsenic cpds.	+		

The BOD Test

The BOD of a sample is a weight of oxygen used in a stated time period. Many people, by inference, use it to indicate the quantity of organic pollution present. Phrases such as "BOD is rapidly removed" and "the conversion of BOD to cellular material" should not be used since that type of thinking can lead to incorrect assumptions about the amount of oxygen required to stabilize organic matter under aerobic conditions. The BOD of a sample is the weight of oxygen consumed by a varying quantity of bacteria in contact with the sample for a specified time and temperature. The test does not differentiate between matter which is biologically oxidized and that which is first synthesized into protoplasm and then oxidized during the endogenous respiration of cellular material. The BOD of a waste will always be less than the theoretical ultimate BOD as some of the cellular material is relatively resistant to microbiological oxidation. Whether the BOD includes the oxygen demand due to nitrification will depend on the sample, seed and duration of the test.

The main limitation of the BOD test is in applying test results to predicting what will take place when the waste is actually oxidized in a different environment. The BOD test is a measure of avidity for oxygen under specified conditions. It does not necessarily follow that the same conditions exist in a waste treatment plant or a stream. In spite of the imperfections and difficulties, the biochemical oxygen demand test is useful in measuring easily oxidized wastes in samples requiring more than 3 mg/l of oxygen.

The COD Test

The chemical oxygen demand is the amount of oxygen consumed by a sample when digested with chromic and sulfuric acids. With the proper catalyst all but a few organic compounds are oxidized to carbon dioxide and water by the conditions of the standard test. The COD test does not measure the oxygen required to oxidize ammonia or organic nitrogen.

The main limitation of the test is that the chemical oxidant reacts with both stable and unstable organic matter. Not only does the test not differentiate between inert and biologically oxidizable matter, but it does not provide any indication of the rate at which the biologically active material would be stabilized under the conditions of waste treatment or in streams.

The test has the advantages of requiring only a short time for the determination, and it is not affected by toxic wastes. Used in conjunction with the BOD test, it will give an indication of the type of wastes. If the wastewater contains substances that are difficult for microorganisms to oxidize, the COD value will be much greater than the BOD value.

This procedure is used principally for wastewaters which have a COD value above 10 mg/l. It can be used on samples containing less than this amount to indicate an order of magnitude.

The TOC Test

The most commonly used total organic carbon method involves the rapid combustion of a micro-sample in a stream of oxygen in a heated tube. The carbon dioxide produced is measured with an infrared analyzer at a wavelength specific for carbon dioxide. This method was designed primarily for analysis of true solutions. The particles in wastes must be reduced by blending to around 50 microns for good results or the needle used for injection will act as a filter. The usable sample size is 20 µl, so some difficulties may arise in getting a representative sample with certain wastes. For this reason in biodegradation studies the dissolved organic carbon is usually determined. That is, the analysis is run on a filtered sample. The apparatus is sensitive to about 1 mg/l total carbon, but for quantitative determinations it is desirable to have samples in the 2 to 3 mg/l range. In order to measure only organic carbon, the inorganic carbon must first be removed from the sample. In samples having a very small amount of organic carbon, it is very important to completely purge the sample of inorganic carbon by first acidifying and then purging with nitrogen. Some of the newer instruments permit the separate determination of carbonate-plus-bicarbonate carbon. In this case the decarbonation step is omitted.

The advantages of the method is that a definitive substance is measured in a very short time, and the results are capable of meaningful interpretation in relation to the theoretical ultimate oxygen demand. The disadvantage or limitation of the method is that all organic matter is oxidized. Such organic matter may or may not be metabolized in biological treatment systems or streams. Many things can be oxidized at high temperatures that would not be oxidized in a stream in a reasonable time. For example, activated carbon is usually not considered as exerting a DO demand on a stream, yet it can be oxidized at high temperatures.

Biodegradability Test Methods

Over the years considerable research has been done on evaluating microbial breakdown of organic compounds. Techniques have ranged from Warburg respirometry used by microbiologists, to BOD determinations and laboratory activated sludge units used by sanitary engineers.

The important features and limitations of various methods of testing biodegradability are as follows.

River Die-Away

The river die-away method is a very simple method and has been used by many investigators. The compound to be tested is added to a sample of river water and left at normal room temperature. Periodically, the solution is analyzed for the test compound to establish a die-away or disappearance curve. The curve usually shows a lag or no breakdown for several days and then a rapid removal when the microorganisms have become acclimatized to the compound. The advantage of the river die-away test is that it requires very little equipment, and it gives both the rate and the total degradability of the test material. The rate is only relative because, after the disappearance of the compound if an additional amount is added, it will disappear in less time. This can be repeated several times so that the die-away curve will have a saw-toothed appearance. This effect is depicted in Figure 2.

Another modification that can be used to reduce the lag time and speed the process of acclimatization is to add a small amount of activated sludge to the river water.

The disadvantage of the test is that all rivers are not the same. Some have more microorganisms than others, and the pH and mineral constituent vary. To be able to use this procedure, it is necessary to be able to analyze for the compound. Likely, the method of assay will detect the original compound, but not a slight modification of the compound. Thus, intermediate products could go undetected. In spite of the procedure's inherent drawbacks, it is useful in obtaining preliminary information on the ease of biodegradation. A large number of compounds can be screened in a relatively short time. If river water is fairly clean, the COD or TOC may be useful if a specific analytical method is not available.

Warburg Respirometer

The utilization of oxygen as the degradation proceeds is a general characteristic of aerobic biological systems. The Warburg respirometer is one of many apparatuses that permit oxygen uptake rates to be determined for a biological system. The reasons for its popularity are that it is commercially available, compact, and permits a number of samples to be run simultaneously.

The oxygen utilization is calculated from changes in pressure differences between that in the closed flask and the atmosphere. The carbon dioxide produced in the reaction

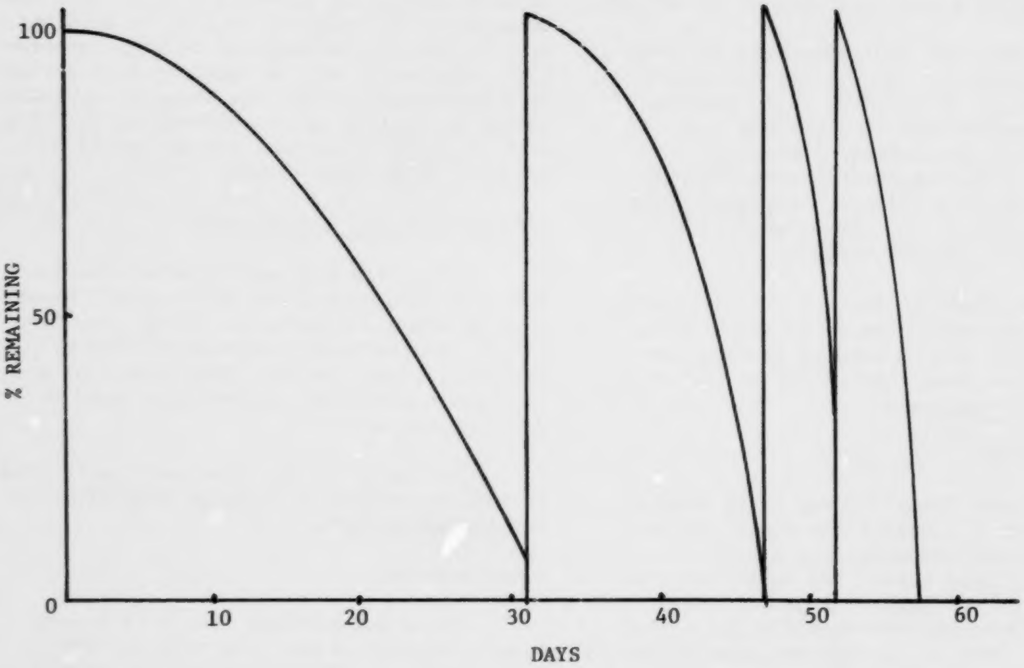


Figure 2. River die-away pattern.

is absorbed in potassium hydroxide contained in the flask. Low results will be experienced if there is not complete absorption of the carbon dioxide. The unit is extremely sensitive to changes in temperature and pressure; therefore, an extra reaction flask is used as a thermal barometer.

There are almost as many variations in operating procedures as there have been users of the apparatus. The two most widely used methods are the three- and four-flask methods. In the three-flask method, the first flask contains only the microorganisms, the second flask, the microorganisms plus the test compound, and the third flask is the thermobarometer control. In the four-flask system, sewage is included in the testing procedure. In this system, one flask contains only the microorganisms, one the microorganisms and sewage, one the microorganisms, sewage and test compound, and one flask is used as a thermobarometer control. The choice of either of these techniques depends upon whether one wishes to study a system relative to waste treatment or only the biodegradability of the compound. If mechanisms and pathways of breakdown are being investigated, the addition of sewage would complicate the study.

There are a number of variables that can affect the results, but the most important factor is the nature and quantity of the microorganisms used. The seed used should be one that has become adapted to the test compound or at least been subjected to an adaptation period. As with any microbial growth system, the greater the initial number of microorganisms present, the more rapid will be the rate of oxygen utilization.

Some of the advantages and disadvantages are:

Advantages:

1. Direct measurement of biological oxidation.
2. Can be used for all compounds without using a different analytical test procedure. This is a decided advantage when no good analytical method exists.
3. Results are available within a reasonable time.
4. Oxygen consumption, radioactive-carbon dioxide evaluation, and respiration assimilation balances can serve as bases for measuring biodegradability.
5. Permits continuous observation of multiple samples over periods of a few hours or several days.
6. Lag periods and rate changes are readily determined.

Disadvantages:

1. Equipment is fairly expensive and requires a skilled technician to operate it.
2. The small sample size makes it difficult to obtain representative samples of sewage and some industrial wastes.
3. All organic matter added to the system is subject to oxidation, therefore necessitating pure organic compounds for fundamental studies.
4. With extended runs, nitrification can occur giving high, erroneous results.
5. Oxygen uptake results are difficult to interpret quantitatively unless supplemented by other analyses.

This last disadvantage needs further explanation. The data obtained from the Warburg respirometer are the amounts of oxygen used in a set time span. The problem is how to convert these data into a measure of the percentage of the compound that was metabolized or degraded. If one compares the amount of oxygen utilized to the theoretical amount needed to completely oxidize the compound, it is always less. This is because in a biological system part of the organic matter is not oxidized but is converted into cells. Without running a material balance on the solution and the cellular matter, it is impossible to decide how much of the compound was changed into new cellular matter. Making a material balance adds to the time and expense expended on the test procedure.

The two most common methods of interpreting the results are the comparative method and assumptive method. In the comparative method, the amount of oxygen utilized by various compounds is compared with that used by a known degradable compound run under the same test conditions. Although this approach gives useful information, it gives only qualitative data and leaves much to be desired. In attempting to relate the oxygen uptake to a definite numerical figure of biodegradation, a value is assumed for synthesis level that occurred. It is generally assumed, based on past data, that degradation is complete when the net oxygen uptake equals or exceeds 35 to 40 percent of the theoretical oxygen demand of the test compound. This assumption is mainly based on the fact that in waste treatment using the activated sludge process about one-third of the organic matter is oxidized and two-thirds are converted into cell mass. This ratio holds true for an activated sludge waste treatment plant operating on domestic waste at a food to microorganism loading of 35 to 50 g BOD_5/100 g MLVSS. It does not hold true for a high-rate treatment plant where the food to microorganism loading is much higher and the plant is operated in the log phase of the bacterial growth curve. Thus, we see that an assumption of a definite conversion value such as 45 percent can give misleading conclusions unless supported by other analytical techniques.

Biochemical Oxygen Demand Test

The biochemical oxygen demand test (BOD) is one of the older methods of evaluating the biodegradability of substances. It was first published as a standard by the Royal Commission for Sewage Disposal of England in 1912. For many years nothing better was available for determining the strength and controlling the treatment of wastewater. The BOD of a substance is the amount of dissolved oxygen utilized during five days when the sample is stored in a filled bottle in the dark at 20°C. Instead of a single determination, a curve of oxygen uptake as a function of time can be developed if a series of duplicates are run. With the development of reliable oxygen-sensing probes it is now possible to determine the oxygen content at intervals or continuously over the entire test period. Another method of BOD determination is by means of apparatuses of the "Sapromat" type. In this apparatus the oxygen uptake and BOD are determined electronically from the electrical current consumed to generate the oxygen required to maintain the pressure equilibrium in the flask.

Many different types of respirometers have been used but, in that they all are based on the same principle, they will not be discussed here. In general, the modifications have been in determining the amount of oxygen utilized or the amount of carbon dioxide produced.

There are numerous limitations to the BOD test in applying the test results to predict what will take place in a waste treatment plant or in a stream. Only the limitations that are pertinent to the utilization of the method for determining biodegradability will be discussed. The advantages and disadvantages of the BOD test procedure are:

Advantages:

1. The apparatus and testing procedure are very simple. The test does not require a highly trained technician.
2. There is a large amount of BOD data available on compounds with known waste treatability in our present waste treatment systems.
3. The small amount of bacterial cells used and the food-free BOD dilution water minimizes interferences with chemical analyses either for the parent compound or intermediate breakdown products.
4. Carbon dioxide is present throughout the test period and does not become limiting as is possible in the Warburg procedure. It has been reported that complete absence of carbon dioxide inhibits assimilation of organic matter by bacteria.
5. Problems of aeration and oxygen transfer are not encountered.

Disadvantages:

1. The amount of sample that can be used is small because of the limited solubility of oxygen in water unless oxygen is replenished as in some apparatuses.
2. The testing period is long although some indication can be obtained in less time if the oxygen uptake is measured periodically.
3. As with the Warburg procedure, nitrification and extraneous organic matter can lead to erroneous conclusions.
4. The interpretation of the results has the same limitations as with the other types of respirometer methods in that the results are not directly convertible to percentage of the compound degraded.

Flask Test Method

This is the most widely used technique of all the laboratory procedures except for the activated sludge. There are many versions of the test procedure but most use a pre-adapted seed and a chemically defined medium. The major number of the test procedures use BOD dilution water as the culture medium. BOD dilution water has been in use by the sanitary engineers for many years. It consists of a dilute solution of calcium chloride, magnesium sulfate, ammonium chloride, ferric chloride, sodium phosphate and potassium phosphate in distilled water. The flasks may be aerated by continuous shaking on a mechanical shaker, agitated with magnetic stirrers, aerated through a ceramic filter, stirred with a paddle stirrer or merely left quiescent. If the medium is not loaded with organic matter, the air can furnish enough oxygen for growth without supplemental aeration. If the static procedure is used (19), the flasks should be mixed every two or three days to prevent the bacteria from settling to the bottom. The test mixture can be analyzed each day or at the end of the specified time period. If degradation is not evident or toxicity is evident, the test compound should be run at a lower concentration. Parallel runs with known degradable compounds are advisable to insure that the test conditions were correct. It is suggested that a compound of similar structure and class serve as a reference standard.

The popularity of the flask test method is exemplified by the fact that five leading countries have adopted some version of the method as their screening or presumptive test method for determining the biodegradability of anionic detergents.

Some of the advantages and disadvantages of the flask test method are:

Advantages:

1. Does not require elaborate or specialized laboratory equipment.
2. Reproducibility is generally good.
3. Sample size is not limited.
4. The test compound does not have to be pure.
5. Can study metabolic products without too much interference if mineral salts are used and the test compound is the sole carbon source.

Disadvantages:

1. Does not show the time required for adaptation if an adapted culture is used.
2. Requires a separate adapting process if adaptation is not made a part of the test procedure.
3. Shaking on a mechanical shaker limits the amount of sample that can be run.
4. Does not simulate any waste treatment process.
5. A chemical or physical analytical test method must be available for each class of compounds tested or a material balance must be run.
6. Simple analytical procedures do not generally measure degradation residues.

Activated Sludge Method

The activated sludge process is one of the most important wastewater treatment processes. Laboratory scale models of the process have been used for many years to study the treatability of various industrial wastes. It was natural that the method should become a major tool in biodegradability testing.

The laboratory methods based on the activated sludge principle vary in feed, residence time, and whether the feed is continuous or batchwise. The continuous activated sludge system is preferred because it corresponds more nearly to practical conditions. A system which approximates the operation of a plug flow aeration tank was developed by Butterfield back in 1937 (20). It is known as the fill-and-draw or the semicontinuous process. In this method of testing, the bacteria inoculum, the test material, and feed are placed in an aerated vessel. The feed is usually a chemically defined medium. The mixture is aerated for a period of time, usually 23 hours. The air supply is then turned off and the mixture allowed to settle. A portion of the supernatant is removed and replaced with new feed containing the test compound. This procedure is then repeated for the duration of the test. If the test period is of long duration, it is necessary to periodically waste some activated sludge in order to maintain a reasonable level of bacteria. In all aerobic biological processes, there is a net increase in bacterial cells.

Although each cycle in the semicontinuous operation constitutes a batch run, the method closely approaches plug flow conditions in a conventional activated sludge plant. The aeration basin of the activated sludge process is usually long and narrow. The recycle sludge and the wastewater enter at one end of the aeration tank. Ideally, the mixture moves as a plug the entire length of the tank. The bacteria are subjected to all phases of the bacterial growth curve from the log phase to the endogenous stage. That is, at the head end of the tank the bacteria have an ample supply of food upon which to feed. It is in this portion of the tank that the bacteria are growing most rapidly and, consequently, the rate of removal of the organic matter is the greatest. As the plug moves along the tank, the food becomes depleted and, if the retention in the basin is correct, the bacteria will start to enter the endogenous phase just as the plug reaches the end of the aeration basin.

The laboratory activated sludge units used by investigators have varied from a few hundred milliliter capacity to several gallons. It depends on the nature of the study. The volume should not be too small so that sampling will disturb the system. For biodegradability testing it is not necessary to maintain a large unit. Small units are much more desirable because of space, cost, and labor of servicing several large units. On scaling down to very small units, there is some difficulty in designing settling chambers. In most units the settling chambers are an integral part of the aeration tank rather than being separate units. The majority of the laboratory units are of the completely mixed type rather than plug flow (21,22).

A list of some of the advantages and limitations are as follows:

Advantages:

1. The continuous and the semicontinuous methods simulate in the laboratory the treatment that wastewater normally receives.
2. There is an accumulation of knowledge over the years since the method has been used for some time.
3. Reproducibility is very good if the unit is operated until steady state is achieved.
4. Sample size is not limited.
5. The test compound does not have to be pure.

Disadvantages:

1. Continuous operating units require money, space, and maintenance.
2. Single batch runs do not allow for acclimatization unless the test period is unduly prolonged.
3. Residence time of 23 hours in the fill-and-draw method is not typical of treatment plants and, therefore, is not realistic.
4. Most investigators feel that valid results cannot be obtained in less than ten days. There are always unexplainable fluctuations in the efficiency of the process that must be averaged.
5. A chemical or physical analytical method is preferable for the analyses of each compound if wastewater is used as feed. BOD, TOC, or COD tests can be used to calculate a material balance and to obtain an indication of breakdown if parallel units are run as controls. True parity of two units is difficult to achieve.

Trickling Filter Method

Most small towns with sewage treatment use trickling filters as the means of secondary treatment. The name is a misnomer to the extent that it is not a filter in the ordinary chemical-engineering connotation. The principle of the method is that the supernatant from the primary settling tank is sprayed upon a bed of crushed stones about 6 feet deep. Recently, plastic trickling filter medium has come into being. The wastewater trickles through the bed to underdrains which lead to a final settling tank. The action of the filter consists in mechanically removing the lighter solids not removed by sedimentation and in the oxidation of the sewage by contact with the microbial film on the filter medium.

Experimental trickling filters have never been a popular device for measuring biodegradability. Small units require small size media which increase operational difficulties due to ponding, air circulation, and side wall effects. If recycle is not practiced, the most important dimension is the depth of the filter bed in that it determines the retention time. The depth of the bed should be at least 6 feet. The main limitation is the time required to develop, acclimatize, and equilibrate the biological film. It can be a matter of months before steady state is reached. If a trickling filter is used for biodegradability studies, the BOD or COD or TOC removals also should be determined so that a comparison can be made between the percentage removal of the other constituents of the wastewater and the test compound.

PHYSICAL-CHEMICAL TREATABILITY

Biological treatment processes are capable of removing essentially all soluble biologically degradable organics found in municipal wastewater. However, municipal wastewaters may contain metals and refractory organic substances. The term refractory is used to mean non-biologically degradable. Secondary biological treatment processes provide some reduction in heavy metals with the activated sludge process being more efficient than the trickling filter process. The amount removed will vary depending on the metal and its form, but in general, 40 to 50 percent of most metals are removed by secondary treatment. High pH coagulation forms insoluble hydroxides from many metals and results in substantial removals of heavy metals. Activated carbon will also remove certain metals from wastewater. The mechanism has not been clearly identified, but it appears that organic compounds may serve as coadsorbates or that metals are adsorbed by the biomass that grows on the activated carbon. Some metals are precipitated by hydrogen sulfide that is present in most columns. Concern for toxicity of heavy metals to human beings is reflected in the 1975 interim United States drinking water standards (23). The recommended levels of several metals are given in Table 5. Future trends indicate that the discharge of certain metals to surface waters will not be permitted.

TABLE 5. METAL CONCENTRATIONS - USEPA INTERIM PRIMARY DRINKING WATER REGULATIONS

Contaminant	Level (mg/1)
Arsenic	0.05
Barium	1.
Cadmium	0.010
Chromium	0.05
Cyanide	0.2
Lead	0.05
Mercury	0.002
Selenium	0.01
Silver	0.05

There are a rather limited number of unit processes which are capable of removing refractory organic matter from wastewater. Adsorption on powdered or granular carbon is the most widely used process. The carbon adsorption process is usually used after biological treatment or after coagulation if the treatment process consists of only physical-chemical treatment.

Coagulation and carbon adsorption are the most commonly used physical-chemical unit processes, with the exception of filtration, in treating wastewater. Laboratory methods for determining if a compound or industrial waste is removed from wastewater by coagulation and activated carbon are summarized.

Coagulation

There does not seem to be a uniform definition of the two terms coagulation and flocculation. The usual definition of coagulation is destabilization and initial aggregation of colloidal and finely divided suspended matter by the addition of a floc-forming chemical or by biological processes. The agglomeration of colloidal and finely divided suspended matter after coagulation by gentle stirring by either mechanical or hydraulic means is commonly referred to as flocculation.

Coagulation is employed for removal of waste materials in suspended or colloidal form. These particles do not settle out on standing and cannot be removed by conventional sedimentation processes. The coagulating chemicals in widest use are aluminum sulfate (alum), ferric sulfate, and chloride. Aluminum sulfate appears to be more effective in coagulating carbonaceous wastes while iron sulfate is more effective when a considerable quantity of proteins is present in the waste. Because wastewaters vary and fluctuate widely in quality, whether coagulation is usable in treating certain wastes is best determined by trial. The usual test method is the "jar test" performed in the laboratory. It is a valuable tool in evaluating different coagulants for determining the optimum dose and pH. Opinions vary considerably as to the best method of performing them and the validity of the result obtained from them. Quantitative information on floc quality is best obtained by other means.

Many try to adopt a procedure that approximates the conditions of the waste treatment plant; others use conditions that are merely convenient in terms of time. It is best to choose a procedure that is suitable and not to depart from it, otherwise it will be impossible to relate the results of coagulation tests made at different times. The equipment needed is fairly simple, consisting of a multiple stirring apparatus with a variable speed motor and any turbidity measuring device.

Although there is no standard jar test procedure, all methods depend on a rapid and complete diffusion of the coagulant as soon as it is added to the test sample (24, 25,26,27). This is followed by a slow and even stirring sufficient to form an easily settled floc and not excessive enough to prevent good floc formation. After several minutes of slow stirring, the stirrer is removed from the test sample and the sample let settle. A sample of the supernatant is carefully removed for testing of various parameters such as solids, BOD, TOC, metals and for specific compounds. The coagulant solutions are usually made so that each milliliter of the solution when added to the volume of the sample will equal a definite number of mg/l. Several runs need to be made with varying coagulant dosages and different pH conditions to determine the optimum conditions and minimum dosages. The first tests are usually rough ones to indicate the approximate ranges of dosages and pH values. Further series of tests are then made to narrow the range down to the minimum dosage of coagulant required and the optimum pH value. The advantages and disadvantages of the jar test are:

Advantages:

1. Indicates if the wastes are amenable to coagulation and flocculation.
2. Provides information on optimum dosage and pH.
3. Provides information on settling rates.
4. Can evaluate several coagulants simply and economically.
5. No short circuiting in jar test trials.
6. Can determine rapidly effects of interfering substances.
7. Useful to determine need for coagulant aids, such as polyelectrolytes.

Disadvantages:

1. Not possible to duplicate in a jar test the conditions present in actual plants.
2. Personal judgment and experience needed in analyzing the data.
3. Computed settling velocity values may be too low due to continued rotation of waste in the jars.
4. Variability of actual wastes cannot be duplicated by jar tests.
5. Not possible to duplicate the energy input in a jar test compared to treatment plant.

Activated Carbon Adsorption

Physical adsorption is usually rapid and a condition of equilibrium between adsorbed and dissolved adsorbate is reached soon after contact with the adsorbent. If the adsorbent is a porous solid like granular carbon or the concentration of adsorbate is small, the attainment of equilibrium is somewhat delayed. As a unit operation, adsorption is generally not confined to the removal of single substances. Therefore, competition and rapid adsorption of one substance could conceivably reduce the number of sites remaining available to other substances. The degree to which adsorption will occur and the resulting equilibrium are given by adsorption isotherms which relate the quantity adsorbed per unit of adsorbent to the concentration of adsorbate remaining in solution. The most commonly used mathematical formulation of these isotherms is the Freundlich equation, $X/M = KC^{1/n}$,

where X = weight of substance adsorbed
M = weight of adsorbant
C = concentration remaining in solution
K and n are constants depending on temperature, the adsorbant and the substances to be adsorbed.

Values of n are normally greater than unity, suggesting that adsorption is relatively more efficient at low concentrations. Although the Freundlich equation can be used in comparative tests to identify the magnitudes of the pertinent adsorption coefficients of a given adsorbent, the equations are seldom fitted to observed information in practice. Instead, dosages are plotted against observed residual concentrations of the sorbate.

At one time the adsorptive capacity of different carbons for pure phenol was used for their general comparison. The phenol value or the amount of carbon in milligrams per liter required to reduce 100 μg/l of phenol 90 percent does not necessarily reflect the relative efficiency of a given carbon in removing other substances.

To evaluate activated carbons for a specific application, a batch test with pulverized samples should be run and the most promising carbon selected on the basis of Freundlich isotherms. To obtain data for scaling up to final plant design, the most promising granular carbons should be tested on small columns under continuous flow conditions. While granular carbons can be compared in their granular form by means of batch test, equilibrium is reached more rapidly in the pulverized form.

There is no standard procedure for determining the adsorption isotherms. In general, different amounts of carbon are added to solutions of the compounds or waste to be tested. The mixture is either shaken or stirred for 1 hour or more, filtered and the filtrate tested for the remaining impurity by an appropriate method. The amount of impurity removed (X) is then divided by the corresponding carbon dosages (M) giving the values $\frac{X}{M}$ which are plotted against the amount of impurity remaining. The adsorption isotherms for the test carbons will show whether the desired final concentration of impurities in the treated wastes can be attained. In addition they will give the relative efficiency of carbons. If each isotherm is extrapolated to the original concentration, the conditions of complete exhaustion in a column will be indicated. The extrapolation is only an approximation of the point of exhaustion or complete breakthrough in column operation.

Advantages of absorption isotherms:

1. Reproducibility is good.
2. Sample size is not limited.
3. Does not require specialized equipment.
4. Test compound does not have to be pure.
5. Shows final concentration obtainable.
6. Gives comparisons between various carbons.
7. Provides carbon capacity.
8. Indicates non-adsorbable fraction of sample.
9. Can be used to determine rate of adsorption.

Disadvantages:

1. Batch runs do not reflect conditions of continuous column operation.
2. Shaking or stirring limits number of samples that can be run at one time.
3. Does not give contact time required.
4. Effect of competitive adsorption of various absorbates difficult to evaluate.
5. Cannot duplicate other reactions which occur in column operation, such as biological activity.

SUMMARY

The emphasis on biodegradable materials which emerged around 1960, relative to detergents, may be expected to continue. In fact, the requirements for biodegradable compounds will increase as the world becomes more conscious of pollution in their waterways. In the United States there is no Federal law or regulation that compounds have to be biodegradable. Several local ordinances have been passed restricting the sale and use of biologically non-degradable detergents. In the future, products which will become a constituent of wastewater will not receive serious consideration for big markets if they are not fully biodegradable.

In general, there is still a lack of agreement on a standard test procedure. In fact, there is not a satisfactory definition of the term biodegradable which is acceptable to all. The question concerning the biological degradability of a compound can only be answered by a biological test. The structure of a compound can be of assistance in predicting the behavior of the compound, but biodegradability depends upon many factors such as size of the molecule, solubility, and the surface activity. Predictions are more reliable when dealing with a homologous series than when making gross comparisons based on functional groups or straight chains.

In determining the treatability of waste, the rate and the extent of degradation must be considered. The rate at which a certain property of a substance changes as a function of time elapsed is not difficult to determine. It is the extent of degradability that poses the problem. There is a need to know if any degradation products are formed in the process as intermediate or end products. The metabolic products, like other nonbiodegradable pollutants, can build up in the surface water supplies, making them undesirable for reuse, or they can build up in food chains causing damage to aquatic life.

The amount of man-made pollution has increased in our surface waters and if it continues, it will become acute. The United States Public Health Service Drinking Water Standards recommend that 200 μg of carbon chloroform extract per liter be the limiting concentration in drinking water. Portions of some of our rivers exceed this amount during low flow. In widespread use of a specific compound, a potential exists for significant environmental pollution if the compound is not degraded within a reasonable time in wastewater treatment plants or rivers. The future would indicate that environmentally acceptable biodegradability for products, which become waterborne wastes, should be that they are completely degraded to products that are harmless in surface and drinking waters.

For wastes that are not amenable to biological treatment, specially designed processes will have to be used to deal with the biologically resistant organic compounds and toxic metals. The wastewater treatment processes in the future will no doubt be a combination of biological and physical-chemical processes. The biological units will be retained for economic removal, and the physical-chemical units will be specifically designed to remove certain constituents.

Several laboratory tests exist for the determination of the treatability of wastes. Hopefully, the future will see progress made in standardization or at least substantial agreement between workers on various test procedures.

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UPDATE OF BIOLOGICAL NITROGEN AND PHOSPHORUS CONTROL

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ABSTRACT

Historically, municipal wastewater plants have been designed and operated to minimize oxygen depletion in the receiving bodies of water and to eliminate disease producing organisms. We are now beginning to be more concerned about the aesthetic quality of natural waters. Consideration is being given to the importance and effects of the addition of phosphorus and nitrogen to streams and lakes. Nutrients attributable to man's activities come from several sources; however, sewage and sewage effluents are a very concentrated source of nutrients. Various nutrient sources and standards that have been established are briefly discussed. The renewed interest in single-sludge nitrification-denitrification processes and biological phosphorus removal are presented. The design and operational data of several treatment plants utilizing these processes are given. Recent research is presented that shows that single-stage nitrification-denitrification is capable of 80-90 percent nitrogen removal at substantial capital and operating cost savings as compared to other nitrogen removal processes.

NITROGEN CONTROL

Nitrogen Sources

Nitrogen is fixed by some bacteria and algae, by natural processes in the atmosphere (e.g. lightening) and by industrial fixation. Annually about 100 million tons of nitrogen are fixed in the world. Most of this is by bacteria on the land and algae in water. The world production of industrially fixed nitrogen amounts to about 33 million tons per year. This amount does not include nitrogen fixed by combustion processes. In the United States alone, 6 million tons of nitrogen per year are fixed by combustion in power plants, heating units, and internal combustion engines. Approximately 11 million tons of ammonia (as N) are produced annually in the United States (1) and over 75 percent of it is converted to fertilizer products or is applied directly to the soil. No more than 2.5 million tons of nitrogen are consumed by other industrial processes in the United States. Few of these nitrogen chemicals are used in a fashion that would be discharged directly to wastewater. However, the discharges from chemical plants with a large waste load of nitrogen chemicals will very frequently have a serious impact on local receiving waters. The strain on the nitrogen balance in nature comes not from fixed nitrogen produced for industrial uses, but from nitrogen fixed for use in fertilizers and nitrogen fixed by combustion processes.

Water pollution by nitrogen compounds generally occurs in the internal recycle between the input and output steps of the nitrogen cycle. The most important source of nitrogen pollution of surface waters is the body waste from people. Municipal wastewaters in the USA average about 20 milligrams per liter of nitrogen, chiefly as ammonium. The average daily contribution of nitrogen per capita is about 12 grams. The 200 million people in the USA thus "process" about 1.0 million tons per year

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of nitrogen. Most of this ends up in our lakes and rivers.

Another impressive large internal recycle stream in the nitrogen cycle is the wastes from our farm animal population. The combined pollutional load from this source is ten times the human load or 10 million tons of nitrogen per year. In the past, most of this nitrogen was deposited on the land where its pollutional impact was not so great. The present trend to feedlots is changing a diffuse loading into a concentrated loading. The nitrogen loading to surface and groundwater from poorly regulated feedlots can be extremely serious.

Nitrogen Standards

Standards for nitrate in drinking water are set primarily to protect against methemoglobinemia. The U.S. Environmental Protection Agency (2) and the World Health Organization (3) have set a limit of 10 mg/l NO_3-N for drinking water supplies. The standard is generally considered sound although some cases of methemoglobinemia have been reported in areas with drinking water nitrate levels less than 10 mg/l NO_3-N and some areas have concentrations greater than 10 mg/l NO_3-N without illness (4). Drinking water standards for ammonia are not normally set (2,3). However, levels of ammonia greater than 0.5 mg/l NH_3-N are undesirable in public drinking water supplies because high ammonia levels indicate recent contamination of the water and interfere with disinfection by chlorine (5).

There are no Federal laws per se that require the removal of nitrogen from wastewater.

The Federal Water Pollution Control Act as amended by the Water Quality Act of 1965 authorizes the Federal and the State Governments to establish water quality standards for interstate waters. The water quality standards submitted by the States are subject to review by the Federal Government. If approved, they become Federal standards, and, if violated, the Government can file a suit to stop pollution to interstate waters. Water quality standards are composed of two parts: the criteria designed to protect present and future water uses of interstate waters and a plan of implementation which outlines the pollution abatement measures.

The Federal Government may also take action to abate pollution of interstate or navigable waters when a situation exists that endangers the health or welfare of persons. The abatement procedure consists of three stages - conference, public hearing, and court action. In a case of violation of water quality standards, direct court action may be sought 180 days from the date of notification of violation. The 180-day period is used for obtaining voluntary compliance if possible.

The requirements for nitrogen removal in the USA have been brought as a result of abatement procedures as described above. Many States, recognizing that 1 mg of ammonia nitrogen can exert an ultimate BOD of 4.5 mg of oxygen, are now requiring 20-day BOD. In this manner, they are indirectly requiring nitrification. Some States have set a limit on the amount of ammonia permitted in effluents. For example, the State of Maryland has placed a limit of 1 mg/l of ammonia in effluents discharged to the Patuxent River. The State of Illinois has recently set an effluent standard of 2.5 mg/l of ammonia for discharge to surface waters.

There are only three locations in the USA where nitrogen removal from wastewater is required. These are the San Joaquin drainage water discharged to San Francisco Bay; Tampa Bay, Florida; and the Upper Potomac River Estuary. For both the San Joaquin drainage and the discharge into Tampa Bay, the total nitrogen removal must be at least 90 percent. The requirement for the Upper Potomac River Estuary is 85 percent removal of the total nitrogen. The municipal wastewaters in the USA have nitrogen contents in the 15-25 mg/l range. Thus, effluent requirements for the above locations would be 2 to 3 mg/l of total nitrogen. The above cases of removal were based on the role of nitrogen as a factor in eutrophication.

Nitrification-Denitrification Processes

Many processes have been explored for the removal of both oxidized and unoxidized nitrogen from wastewater. To mention a few: air stripping, electrodialysis, reverse osmosis, breakpoint chlorination, algae harvesting, land application, ion exchange, and biological nitrification followed by denitrification. Only the latter method will be discussed in this manuscript.

The most common method of removing nitrogen from wastewater is the biological nitrification-denitrification process. The process basically consists of oxidizing all the ammonia to nitrates (nitrification) and then reducing the nitrates to nitrogen gas (denitrification) which is released to the atmosphere. Nitrification alone will remove ammonia nitrogen, thus aiding in eliminating the problem of ammonia toxicity to fish and reducing the oxygen demand exerted by ammonia nitrogen in streams. However, the residual nitrite and nitrate can serve as nutrients for undesirable algae growths in streams and lakes. With proper control of wastewater treatment processes both nitrification and denitrification can be accomplished, thus removing not only ammonia nitrogen but also nitrite and nitrate nitrogen. True biological denitrification occurs only on the oxidized forms of nitrogen. Thus, it is essential to have complete nitrification to assure complete denitrification. This method has the advantage of returning nitrogen to the atmosphere in its natural form.

There are many different ways to combine the nitrification and denitrification processes with a process for organic removal. Some of these are illustrated in Figure 1. In the first system the nitrification occurs in the same tank as organic removal, and the mixed liquor then flows into a separate non-aerated tank where denitrification takes place. The mixed liquor volatile suspended solids are oxidized and serve as the electron donor for the denitrification reactions. Some ammonia is released during oxidation of the suspended solids in the denitrification tank so that removal efficiencies around 50-70 percent are usually obtained unless high recycle rates are employed. If the initial aerobic treatment is carried out efficiently, most of the available carbon should be oxidized or assimilated into new cell protoplasm. Although the activated sludge mass will provide some carbonaceous material from endogenous activity, it is only enough for a low endogenous denitrification rate. Variations of this process were studied by Ludzack and Ettinger (6), Wuhrmann (7), and Johnson and Schroepfer (8).

In the second system, the organic oxidation and nitrification are separated from the denitrification step (9). A variation in which denitrification was coupled with stabilization in a contact-stabilization process was explored by Balakrishnan and Eckenfelder (10). In their approach the nitrified effluent and the solids from the first stage clarifier were mixed in basins for stabilization and denitrification before final clarification. In the two-stage system either primary effluent or an organic chemical such as methanol is used as an electron donor in the denitrification tank. When wastewater is used for this purpose, the nitrogen removal is usually no more than 60 percent because of the carry-over of organic nitrogen and ammonia. Addition of methanol is easier to control and does not add nitrogen, so the removal is higher. The cost of methanol is a factor to be considered.

In the third illustration, the three processes of organic removal, nitrification and denitrification are separated to give better control of each of the unit processes. The total tank volume may be larger than for either of the first two systems. Increased dependability, ability to control each process, greater freedom from influence of inhibitors and higher removal efficiencies are distinct advantages cited by Barth, et al. (11); Downing, et al. (12); Mulbarger (13); and Horstkotte, et al. (14).

In most of the studies, methanol was used as the external source of carbon as a supplement to promote denitrification. Herein lies one of the major difficulties with the biological process in that an oxidizable substrate must be present to serve as an electron donor. The nitrate merely serves as a substitute for oxygen that permits the microbial cell to carry on its metabolism in the absence of oxygen. A wide range of organic compounds have been used in heterotrophic denitrification processes: ethanol, diethyl ether (15); lactate (16); volatile acids (17); molasses (18); sucrose, acetate and acetone (19). Mixtures of organics in industrial wastes have also been used: skim milk, carboxylic acids (8); chicken manure (20); brewery waste (21); and an unidentified industrial waste (22). Until recently methanol was the compound of choice as a carbon source due to its relatively low cost and low biomass-yield (19). The cost of methanol has increased dramatically. Between July 1973 and July 1975 the price of bulk methanol increased 190 percent (23,24). The price of methanol on January 3, 1977, was \$.43 a gallon on the Gulf Coast and \$.53 on the West Coast of USA.

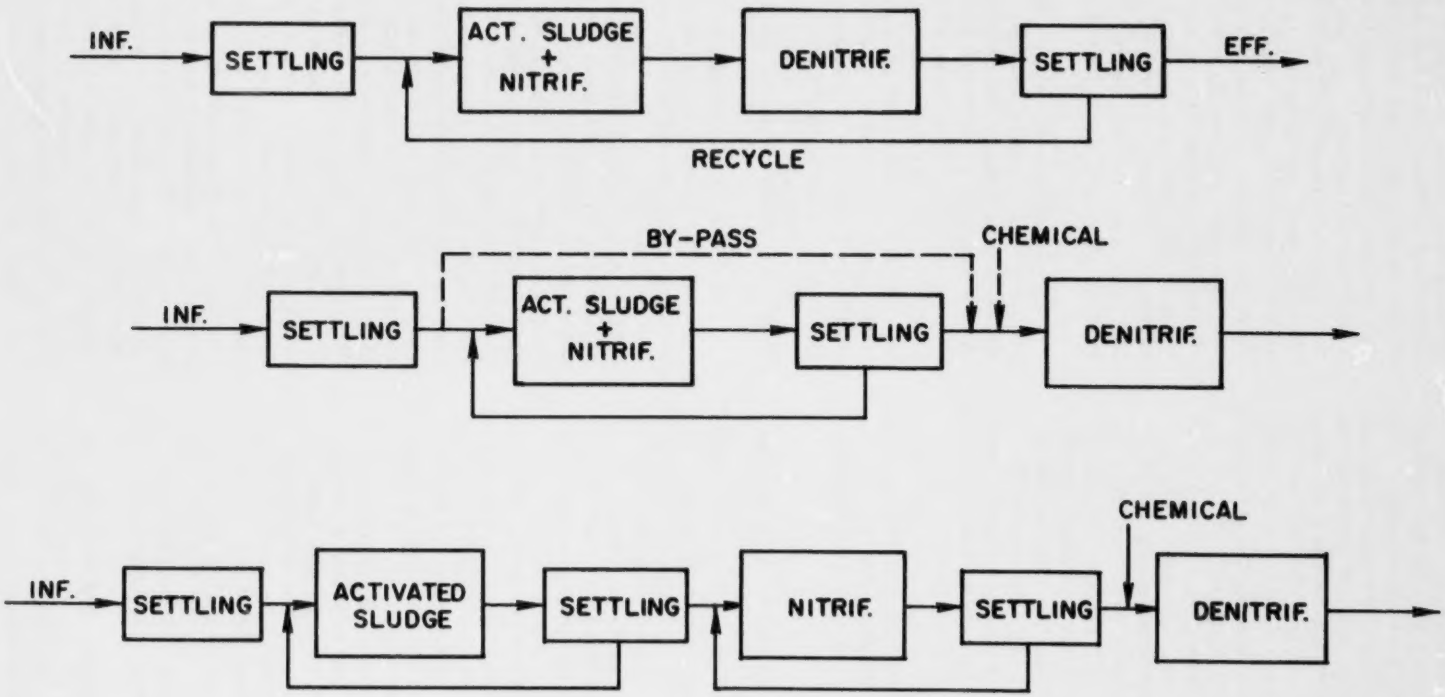

Figure 1. Flow sheet for various nitrification-denitrification biological systems.

In the previously discussed heterotrophic denitrification systems, various forms of carbon were used. An alternative to heterotrophic biological nitrate removal is the use of Thiobacillus denitrificans in an autotrophic denitrification process. This microorganism is responsible for denitrification in soils and in wastewater treatment systems when reduced sulfur compounds are available. This organism does not require organic compounds and can reduce nitrate to nitrogen gas while oxidizing a wide variety of sulfur compounds to sulfate. The U.S. Environmental Protection Agency has funded a grant (No. 803505) to Cornell University, Ithaca, New York, to study the technical feasibility of this process. The Principal Investigator is Dr. Alonzo W. Lawrence. The objective is to retain the advantages of biological processes without incurring the high cost of petrochemicals. The economic feasibility of autotrophic denitrification using sulfur will depend on the cost of sulfur.

Sulfur is presently relatively inexpensive and widespread adoption of sulfur oxide removal technology for stack gases would mitigate against any future price increases. The amount of sulfur which could be made available from sulfur oxide control is significant. In 1968, emissions of sulfur from power plants were estimated as 12.2 million tons, while the total U.S. sulfur production was 10.4 million tons.

The relative cost of various sulfur substrates depends on the initial cost of purchase and the amount of the compound required for nitrate removal. The exact amount is not known, but a reasonable comparison can be made on an electron equivalent basis, since the nitrate removal is brought about by an oxidation-reduction reaction. Table 1 shows a comparison of various compounds and methanol expressed on an electron equivalent basis. Elemental sulfur is the least expensive and has the advantage of ease of storage and handling. From a cost standpoint, hydrogen sulfide is favorable, but storage and handling would present some serious health and odor problems.

TABLE 1. COMPARISON OF COSTS OF VARIOUS SULFUR COMPOUNDS AND METHANOL(25)

Compound	Unit Cost ($/100 lbs)	Cost/Electron Equivalent (¢/electron equivalents)
Sodium sulfite, anhydrous	11	1.53
Sodium thiosulfate, pentahydrate	8.30	0.567
Sodium thiosulfate, anhydrous	10.95	0.477
Sodium sulfide, flake	12.50	0.826
Hydrogen sulfide	10	0.094
Sulfur, crude flour	4.70	0.055
Methanol	7.59	0.089

lb = 0.453 kg

The ultimate feasibility of any wastewater treatment process depends on its economic, social and environmental costs relative to alternative processes. Bench-scale results have shown the process is technically feasible. Much remains to be done before the method can be considered a viable process. Autotrophic denitrification processes have certain disadvantages. They enrich the wastewater in sulfate and destroy alkalinity.

Single-Sludge Systems for Nitrogen Removal

Nitrogen removal facilities can cost two to three times to construct and operate as compared to the conventional secondary treatment. This high cost is largely due to the fact that presently the most reliable biological system necessitates three separate stages of activated sludge treatment with different types of bacteria employed in each of the three stages. Additionally, methanol must be added in the denitrification stage. In the past two years, the energy situation has resulted in higher energy and methanol costs. Sanitary engineers are faced now with a trend toward conservation of resources and improvement of effluent quality. For this reason there has been a renewed interest in single-sludge systems. Recent research has indicated that three different bacteria can exist and perform their proper function in a single-stage system under properly controlled operating conditions. A brief review of the three classes of organisms required will suffice since they have been adequately covered in the literature. Aerobic heterotrophs provide carbonaceous removal by microbial assimilation and oxidation. Aerobic autotrophs, collectively known as nitrifiers, provide the oxidation of nitrogen in a two-step microbial reaction: first, ammonia is converted to nitrite by *Nitrosomonas*; then, nitrite is converted to nitrate by *Nitrobacter*. Facultative heterotrophs provide denitrification by nitrate respiration, i.e., the microbial reduction of nitrate to nitrogen gas.

The design and performance of some of these single-sludge systems for nitrogen removal will be reviewed. No one combination of processes will be universally applicable but given a variety of options, the environmental engineer can then develop a cost-effective system to suit the local situation.

Early Studies

As previously cited, Ludzack and Ettinger (6) as early as 1962 used alternate anaerobic and aerobic zones within a single activated sludge reactor to provide for a period of low or zero dissolved oxygen for denitrification. Since the conditions for denitrification cannot be regarded as strictly anaerobic, it is becoming customary to use the term "anoxic" instead of anaerobic. This recognizes the fact that the nitrate nitrogen is the final electron acceptor. In their laboratory-scale system, the effluent mixed liquor, with the nitrate product, from the aerobic (second) zone was recycled to the reactor influent zone which was operated under anaerobic conditions. With ample supply of incoming carbonaceous material, the nitrate was reduced to nitrogen gas during the biological oxidation of the carbonaceous material. The amount of denitrification was controlled by the rate of recycle of the mixed liquor from the aerobic zone to the anaerobic zone.

Oxidation Ditches

Pasveer was the first to investigate denitrification in oxidation ditches and reported removals approaching 90 percent. Few details have been reported on his studies (26,27). However, subsequent investigators have confirmed his results and defined the conditions required for denitrification in oxidation ditches. Even though aeration devices have ranged from Kessner brushes and cage aerators to vertical turbine mechanical aerators, all the variations can be termed oxidation ditches because they all have a channel with aeration devices placed at points in the channel. Figure 2 is a typical layout. By controlling the amount of aeration, the mixed liquor is exposed to alternating aerobic and anoxic zones. The channel operates as an endless channel, so to speak, since only a portion of the mixed liquor passing the outlet is withdrawn. Thus, the mixed liquor is recirculated many times through aerobic and anoxic zones prior to discharge from the channel.

Matsché (28,29) reported significant removals of nitrogen in an oxidation ditch type activated sludge plant at the Vienna-Blumental Works situated south of Vienna. This is the largest oxidation ditch plant (1.0 m^3/sec) that has been employed for nitrogen removal testing. The plant consists of a pumping station, screens, aerated grit chamber, two aeration tanks, and two secondary sedimentation tanks. The plant does not utilize primary sedimentation. The flow of the plant is shown in Figure 3. The design data are given in Table 2.

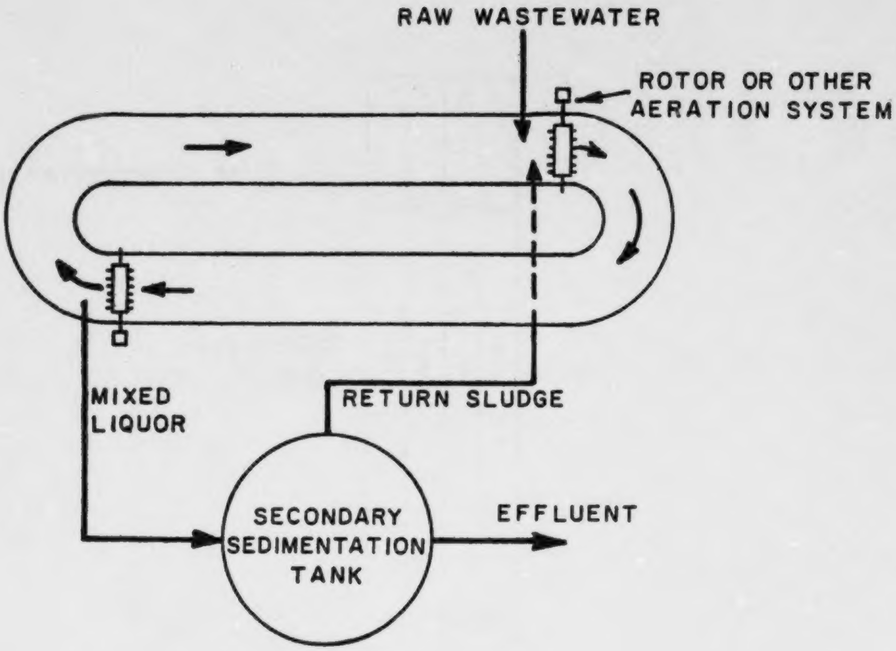

Figure 2. Pasveer ditch system for nitrogen removal.

TABLE 2. VIENNA-BLUMENTAL TREATMENT PLANT DESIGN DATA(29)

Component	Value
Population equivalents (65 g BOD/d)	150,000*
Average dry weather flow (ADWF)	1.0 m^3/sec
Aeration tanks	
Total volume	12,000 m^3
Width (each)	17 m
Length (each)	150 m
Depth	2.5 m
Final sedimentation tanks	
Diameter	45 m
Depth	3.0 m
Overflow rate ADWF	27.2 $m^3/m^2 \cdot d$

*1st step of completion; ultimate 300,000.

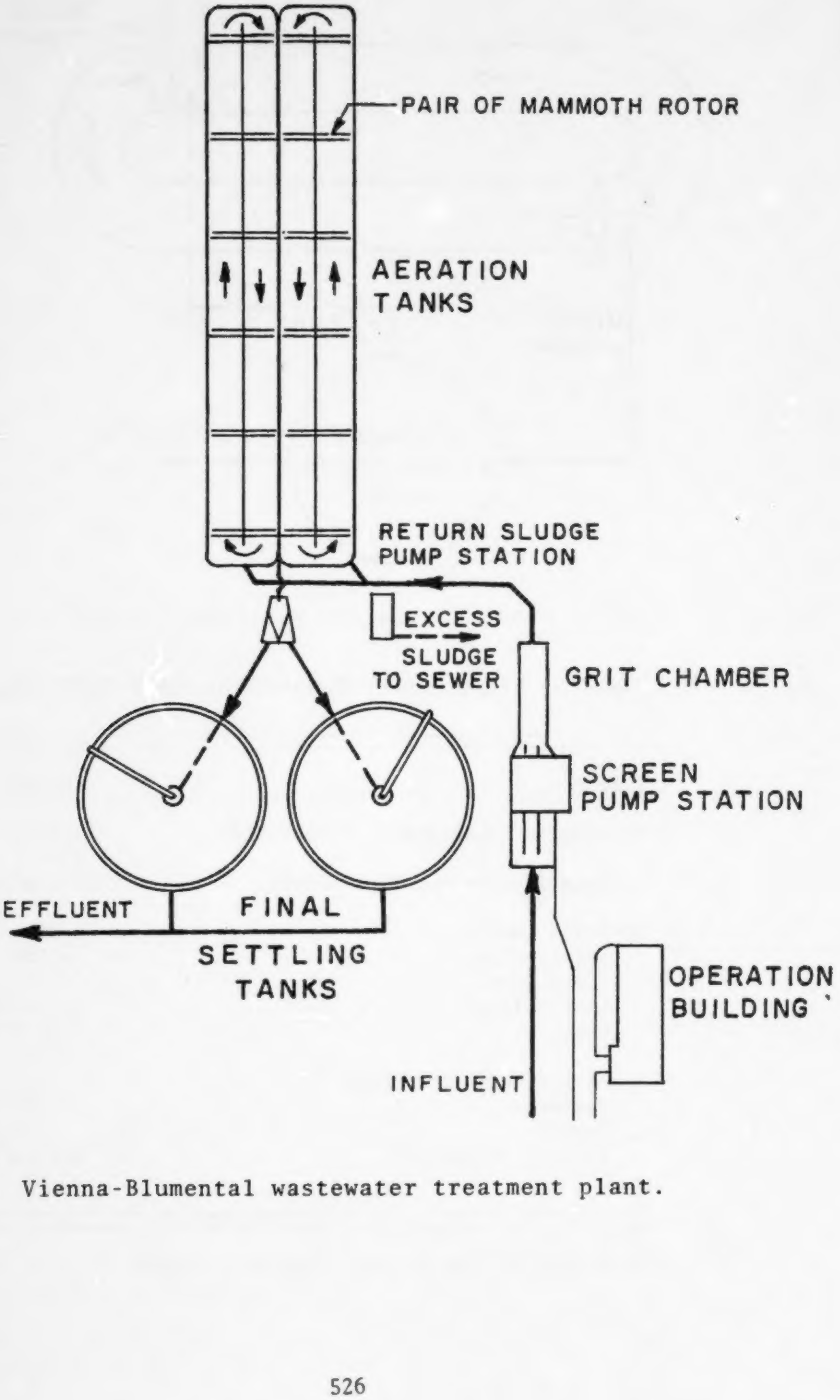

Figure 3. Vienna-Blumental wastewater treatment plant.

During the investigation for nitrogen removal, the two aeration tanks were connected in series. The dissolved oxygen concentrations immediately after the rotors were above 2 mg/l, but dropped to near zero before the next rotor. This resulted in an alternating contact of the mixed liquor with aerobic and anoxic zones. Nitrification took place in the aerobic zones, and denitrification occurred in the anoxic zone. The results of special 24-hr tests, as shown in Table 3, indicate that ammonia nitrogen removals of 80 to 90 percent were obtained. The operating conditions deserving special attention are the detention time of 5.5 to 8.4, high MLSS of 5,300 to 6,700 mg/l and low overflow rates of 11.4 to 16.6 $m^3/m^2 \cdot d$. Higher overflow rates in the final sedimentation would probably result in the inability to maintain high MLSS levels under aeration, therefore the aeration time would need to be higher.

It was observed by Drews, et al. (30) that "orbal" extended aeration plants usually produced effluents containing less total nitrogen than those from conventional activated sludge plants. This prompted him to explore the possibility of further increasing the nitrogen removal by alternating the channels between aerobic and anoxic conditions. The plant used in his investigation has been described in detail (30); therefore, a brief outline only will be given. The plant consisted of four concentric aeration channels in series, a settling tank, and disc aerators 1.2 m in diameter perforated with 1.2 cm holes. The oxygen input was adjusted by the number of discs used. A schematic of the plant is shown in Figure 4.

TABLE 3. OPERATION AND PERFORMANCE DATA
VIENNA-BLUMENTAL PLANT(29)

Operational Data		Sept 2 1971	Feb 17 1972	June 6 1974	July 16 1974	July 31 1974	Aug 7 1974
Wastewater flow, m^3/sec		0.42	0.61	0.54	0.48	0.42	0.40
Aeration water temp., °C		18	12	18	20	19	20
Aeration detention time, hr		8.0	5.5	6.1	7.0	8.0	8.4
MLSS, g/l		6.7	6.2	5.3	5.9	5.4	5.4
Return sludge SS, g/l		10.0	10.3	8.0	9.0	7.9	8.4
Sludge loading (F/M)		0.11	0.13	0.20	0.19	0.16	0.24
Operating rotors, tank 1, tank 2		4, 2	3, 3	3, 3	4, 2	4, 3	4, 4
Clarifier overflow rate, $m^3/m^2 \cdot d$		11.4	16.6	14.7	13.0	11.4	10.9
Performance Data							
BOD$_5$	influent, mg/l	268	200	268	-	-	-
	effluent, mg/l	13	13	10	-	-	-
	removal, %	95	93	96	-	-	-
COD	influent, mg/l	475	384	463	575	515	778
	effluent, mg/l	49	50	39	39	35	58
	removal, %	90	87	92	93	93	92
TOC	influent, mg/l	155	126	128	143	134	208
	effluent, mg/l	14	13	12	14	13	15
	removal, %	91	90	91	90	90	93
Ammonia-N	influent, mg/l	21.8	9.0	21.8	16.6	17.2	21.2
	effluent, mg/l	3.8	2.7	6.3	3.9	2.4	2.7
	removal, %	83	70	71	76	86	87

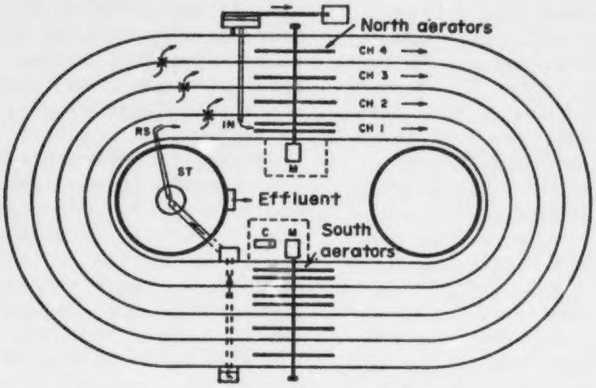

Figure 4. Orbal extended aeration plant (Mode #1).

During the period of investigation the plant was operated in four different modes. The number and arrangement of the discs are shown in Table 4.

TABLE 4. AERATION DISC CONFIGURATION

Channel	Mode			
	1	2	3	4*
	Number of Aeration Discs			
1	4	1	4	2
2	3	5	1	1
3	2	1	4	1
4	2	3	1	1

*Also other variations.

Mode 1 was the conventional operation of the plant. Modes 2 and 3 were designed to give nitrification and denitrification in alternate channels. This type of operation produced a layer of floating sludge in the settling tank. Mode 4 gave the best operation. Data for two summer runs and two winter runs are presented in Table 5. For additional information, consult reference (31).

Christensen (32) proposed a novel alternating contact process which had been tested in Denmark on both laboratory and full-scale. It consisted of two basins and a clarifier. The operation had four operational sequences as depicted in Figure 5. The process employed alternating aeration in first one basin and then the other to produce the alternate high-low oxygen concentration needed for nitrification-denitrification. The mixed liquor was only discharged to the sedimentation tank when it was under aeration. Reportedly total nitrogen levels of 2-5 mg/l were obtainable under proper operating mode.

Bardenpho Process

A single reactor-clarifier system using four basins in series, alternating from anoxic to aerobic was developed by Barnard (33,34). The process was termed the "Bardenpho" process by its developer. The mixed liquor containing nitrate is recycled from the second (aerobic) tank to the first (anoxic) tank for denitrification. Following the first two tanks is a third (anoxic) tank for removing the nitrate remaining in the effluent from the second (aerobic) tank. In this third tank endogenous respiration is used for denitrification. The final basin is used to strip the N_2 to improve the sedimentation. A schematic of the process is shown in Figure 6.

A 9-month test was conducted in a 100 m^3/d pilot plant. Nitrogen removals in the range of 80-90 percent were achieved during the last three months. The process tends to produce a bulking sludge.

TABLE 5. OPERATING CONDITIONS AND PERFORMANCE FOR MODE 4(31)

Operation	Summer		Winter	
Feed, l/h	4800	8000	500	6000-6500
Retention time, h	23.2	13.9	22.2	17.1-18.5
Return sludge ratio	2:1	1.5:1	2:1	>2:1
MLSS, mg/l	4000*	4000*	3300-4000	3300-4100
Water temp., °C	21-22.5	20-22	14-17	14-15.5
SVI, ml/g	213	212	263	268
COD load, COD/MLVSS	0.155	0.28	0.20	0.26
Performance*				
Influent				
COD, mg/l	749	791	678	723
Kjeldahl-N, mg/l	39.5	39.4	43.1	45.0
Ammonia-N, mg/l	21.3	20.7	28.2	29.2
Effluent				
COD, mg/l	28.2	30.9	34.4	37.4
Kjeldahl-N, mg/l	4.5	4.05	6.4	5.0
Ammonia-N, mg/l	3.4	3.3	3.4	2.1
Nitrate-N, mg/l	1.4	2.65	3.4	7.1
COD removal, %	96.2	96.0	94.6	94.5
N-removal, %	85.2	81.6	77.3	72.0

*Average results.

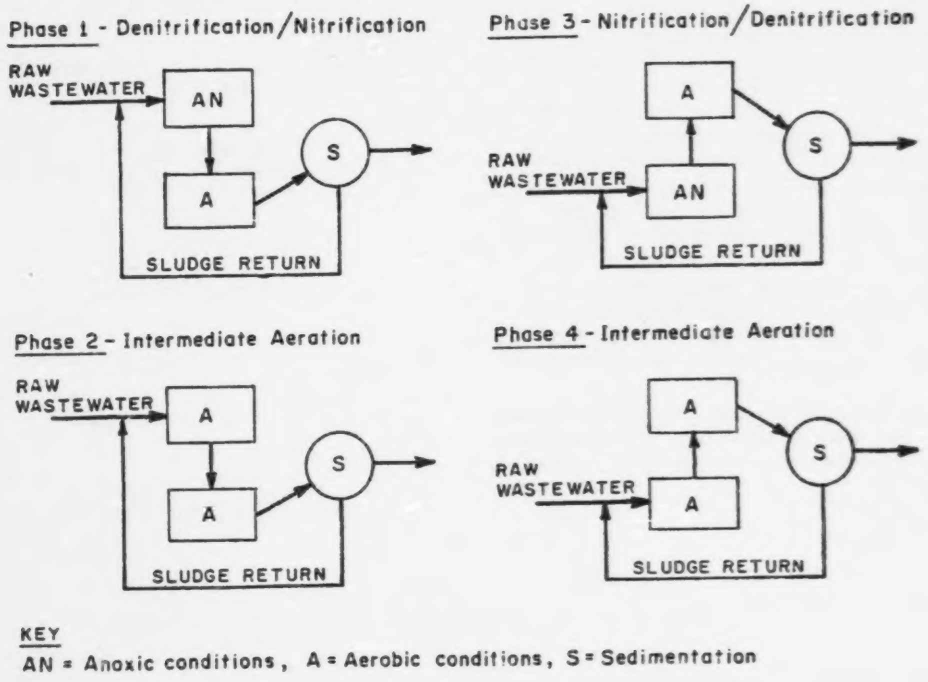
Phase 1 - Denitrification/Nitrification
RAW WASTEWATER
AN
A
S
SLUDGE RETURN
Phase 3 - Nitrification/Denitrification
A
AN
S
RAW WASTEWATER
SLUDGE RETURN
Phase 2 - Intermediate Aeration
RAW WASTEWATER
A
A
S
SLUDGE RETURN
Phase 4 - Intermediate Aeration
A
A
S
RAW WASTEWATER
SLUDGE RETURN
KEY
AN = Anoxic conditions, A = Aerobic conditions, S = Sedimentation

Figure 5. Operational sequence of alternating contact process.

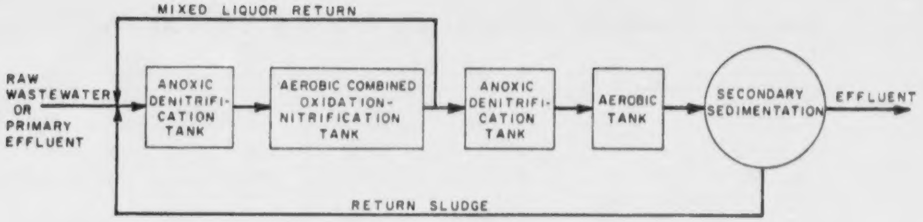
MIXED LIQUOR RETURN
RAW WASTEWATER OR PRIMARY EFFLUENT
ANOXIC DENITRIFICATION TANK
AEROBIC COMBINED OXIDATION-NITRIFICATION TANK
ANOXIC DENITRIFICATION TANK
AEROBIC TANK
SECONDARY SEDIMENTATION
EFFLUENT
RETURN SLUDGE

Figure 6. The Bardenpho process.

Blue Plains Pilot Plant

The removal of 49 to 84 percent nitrogen from primary wastewater has recently been achieved in a single-stage activated sludge process at the USEPA Blue Plains pilot plant (35,36). The 189 m^3/d pilot plant contained a two-pass aeration tank provided with separate aeration and mixing facilities. Air was supplied alternately to each basin in a 30-minute cycle. The basins were alternately aerobic and anoxic. Each basin had two mechanical mixers to keep the mixed liquid in suspension. A flow diagram is shown in Figure 7.

The reactor was operated with a food to mass ratio of approximately 0.1 (BOD_5/MLVSS). At this F/M ratio good nitrification and denitrification occurred. The COD/TKN ratio in the wastewater entering the reactor controls the amount of nitrogen removal. With a COD/TKN ratio of 10, 84 percent removal of total nitrogen, based on the primary effluent, was achieved in the summer and 75 percent in the winter. The detention time was between 9-12 hours with the longer time being used in the colder months. When the COD/TKN ratio was reduced to 7.5, by the addition of $FeCl_3$ treatment up stream of the primary, the nitrogen removal in August dropped to 67 percent. A summary of the operational and performance data are presented in Table 6.

Some bulking of the sludge was experienced in the winter months. This may limit the clarifier overflow rates to 12 $m^3/m^2 \cdot d$.

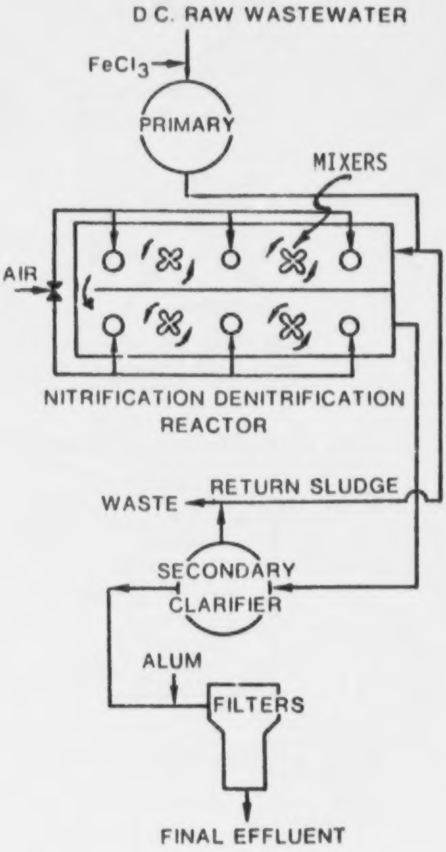

Figure 7. Blue Plains process.

TABLE 6. BLUE PLAINS' OPERATIONAL AND PERFORMANCE DATA (35)

							Influent quality, mg/1			Effluent quality, mg/1[a]				Removals, percent		
Month 1973	Det. time, hr	F/M Ratio BOD/MLVSS	MLSS mg/1	SVI	COD/ TKN ratio	Temp., C	BOD_5	SS	Total Kjeldahl Nitrogen	BOD_5	SS	Total Kjeldahl Nitrogen	NO_3^- & NO_2^--N	BOD_5	SS	Total Nitrogen
Jan.	12.3	0.072	3510	245	9.6	14.0	96.5	110	25.7	20.4	15.4	2.28	3.99	79	85	76
Feb.	12.3	0.066	3980	250	9.9	14.2	99	108	23.2	14.0	14.3	1.52	4.41	86	87	75
March	12.3	0.10	2950	330	10.5	15.5	110	128	24.8	6.5[b]	15.0	4.20	2.30	94[b]	88	74
April	12.4	0.081	3540	277	10.6	-	98.9	120	21.7	5.3[b]	13.0	5.20	6.03	95[b]	89	49
May	10.5	0.089	4170	227	10.0	-	115	109	23.3	3.3[b]	11.8	1.36	8.25	97[b]	89	59
June	8.8	0.105	4010	188	10.3	23.0	107	112	24.0	3.2[b]	7.8	1.51	2.30	97[b]	93	84
July	6.8	0.093	3040	133	7.9	25.0	51	153	15.0	3.8[b]	9.0	2.14	2.72	93[b]	94	68
August	6.6	0.089	3200	134	7.5	25.5	44.2	197	14.9	2.6[b]	10.0	1.23	3.74	94[b]	95	67
Sept.	8.7	0.11	3700	-	10.0	26.0	99	110	22.6	7.2[b]	16.0	10.2[c]	0.22	93[b]	85	54

[a] Prior to filtration.

[b] Nitrification inhibited.

[c] Ammonia level was 9.4 mg/1 as N.

Owego, N.Y., Demonstration

Encouraged by the pilot plant results at Blue Plains, the U.S. Environmental Protection Agency desired to study the operation of a full-scale single sludge nitrification-denitrification plant. There is a need to determine the reliability of the process, develop operating techniques, and identify the best design features to insure optimum performance. For the study, an extended aeration plant at Owego, N.Y., was selected. The town of Owego facilities were very suitable, and they were willing to cooperate with the USEPA in a study. The study is partially supported by USEPA Grant No. 803618.

The Owego Water Pollution Control Plant No. 2 has several desirable features which are conducive to such a study. The nitrification-denitrification process can become more difficult to operate under cold wastewater temperatures; therefore, this plant offers a wide range of temperatures. The wastewater temperature ranges from 8°C to 22°C. The extended aeration activated sludge process has a long aeration time which tends to promote nitrification. The plant also is capable of operating at shorter detention periods, thereby, providing variable conditions for study. Lastly, the plant required very little modification for the study. An additional reason for the selection of an extended aeration plant is that there are many extended aeration plants in the United States. Success of the demonstration would mean a significant improvement in the national water quality could be achieved at a moderate cost.

The Owego Water Pollution Control Plant No. 2 was designed for the year 1990 flow of 7600 m^3/d. The plant became operational in 1971 and presently has a flow of 1900 m^3/d. The wastewater is typically domestic in character. Flexibility was provided in the design so that the plant could be operated as a conventional activated sludge process when the flows increased in the outer years. A summary of the design of the key components is given in Table 7.

The major operating modification was the bypassing of primary settling tanks and putting the raw sewage directly into the aeration tanks. This was done because the Blue Plains study had shown that a high BOD/TKN ratio was needed for effective denitrification. A flow schematic of the treatment, as modified for the study, is shown in Figure 8. To provide for alternating aerobic-anoxic conditions, automatic timers were installed so that the mechanical aerators in compartment N-1 and S-1 could be cycled. The size of the aerators was such that they could be operated on a 50 percent on-off cycle and still supply the required oxygen for carbonaceous oxidation and for nitrification.

The study was divided into two phases: warm and cold weather operation. The two phases were subdivided into periods when all the aeration tank capacity was used and when only half the aeration tanks were in operation. The process loading characteristics and performance are shown in Table 8. The final report on this grant will be available after July 1977.

During the aerobic cycle, dissolved oxygen levels of about 0.7 to 1 mg/l appeared most compatible. Lower concentrations appeared to give incomplete nitrification, while higher concentration delayed the onset of denitrification. Wastewater organics were suitable as a carbon source for denitrification. During the summer months it was necessary to significantly reduce the mixed liquor suspended solids to prevent a deficiency in carbon source for denitrification. This is quite noticeable in the July results of Phase IA. A combination of increased bacterial metabolism rates and a decrease of COD/TKN resulted in a deficiency of organic sources for denitrification. This condition was corrected by removing one aeration tank from the operation. An alternative corrective measure may have been to reduce the aerobic cycle time and increase the anoxic cycle.

Some unforeseen operational problems were encountered during the study. The primary digester had to be shut down for repair and the secondary digester modified to allow it to function as a primary digester. This probably contributed to the rapid deterioration of the nitrogen removal experienced in the last part of Phase II. The plant received an increase in the amount of digested solids in the digester supernatant being returned. These digester solids caused a significant reduction in solids time and a marked change in the nature of the mixed liquor solids which led to a loss of denitrification. Toward the end of January 1976, extremely high flows were received due to infiltration and thawing of snow.

TABLE 7. DESIGN DATA, OWEGO, N.Y.

Component	Value
Bar Rack	
No. of units	1
Capacity	19,000 m^3/d
Comminutor	
No. of units	1
Capacity	19,000 m^3/d
Grit separator (cyclone)	
No. of units	1
Size	0.3 m
Primary settling tanks	
No. of tanks	2
Total surface area	230 m^2
Diameter	12 m
Depth (average)	2.7 m
Aeration tanks	
No. of tanks	2
No. of compartments/tank	2
Volume of compartments	
First (N-1 & S-1)	1,190 m^3
Second (N-2 & S-2)	760 m^3
Depth	3.7 m
Mechanical aerators/compartment	1
Aerator horsepower	11 kw
Return sludge rate	1900-5700 m^3/d
Final settling tanks	
No. of tanks	2
Length	26 m
Width	4.4 m
Depth (average)	2.6 m
Surface (area)	240 m^2
Total volume	630 m^3

This caused an hydraulic washout of solids. Although solids were rapidly reestablished and good carbonaceous removal was established, it took four weeks to reestablish the nitrifying bacteria. The denitrifying bacteria were readily established.

The study indicated that 80 to 90 percent nitrogen removal is feasible with the proper solids inventory, cycle time and oxygen levels. The conditions must be favorable for good nitrification and an adequate carbon source must be present during the anoxic period. To obtain year-round efficient nitrogen removal, careful operational control of solids inventory must be maintained. Final settling tanks must be provided with low surface overflow rates to prevent solids washout at high flows and be sufficiently deep to allow for diurnal blanket level variations without solids carryover.

Before the single sludge nitrification-denitrification process will be accepted by the design profession as a viable alternate to currently used nitrogen removal processes, it will require further testing. Additional data on process reliability, performance and operational control is needed.

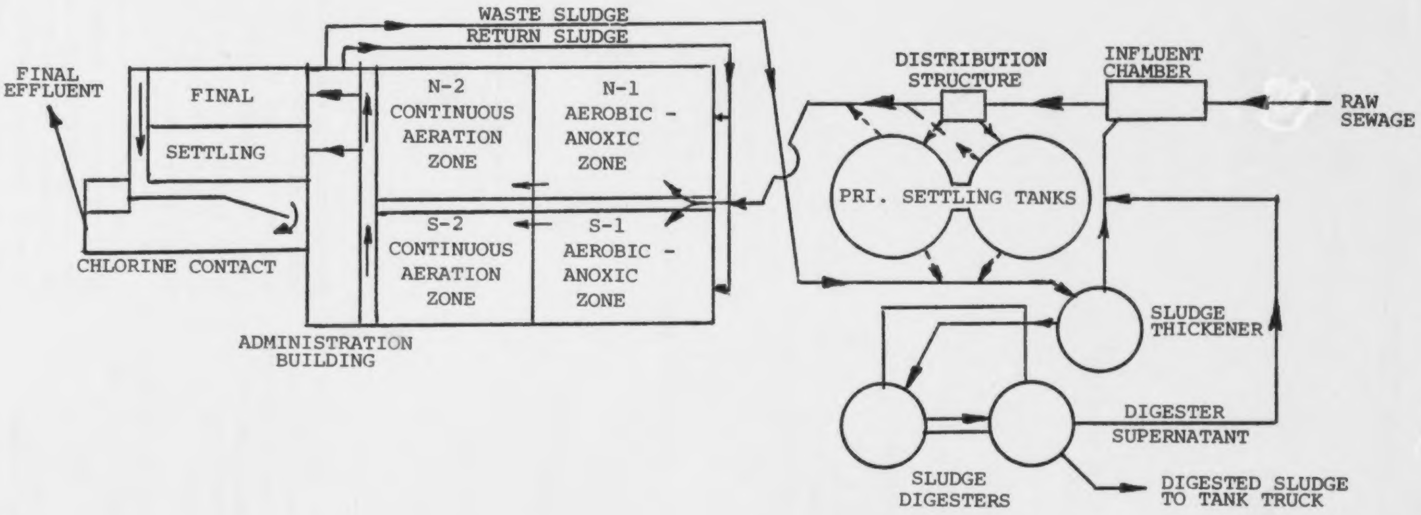

Figure 8. Schematic flow diagram, Owego Water Pollution Control Plant No. 2.

TABLE 8. OPERATIONAL AND PERFORMANCE DATA, OWEGO, N.Y.

	Phase IA-1975			Phase IB-1975			Phase IIA-1976		Phase IIB-1976	
Operational Data	May 5-31	June 1-30	July 1-16	July 17-31	Aug. 1-31	Sept. 1-8	March 10-31	April 1-30	May 1-9	May 10-28
Wastewater flow, m^3/d	1830	1730	1430	1480	1650	1750	2380	1990	2010	1980
MLSS, mg/l	3340	3260	2660	3250	3810	3990	3860	3020	3200	3020
SRT, d	69	41	44	19	18	13	32	20	12	10
Aeration tank HRT, h	25.7	27.3	32.8	15.9	14.3	13.4	19.8	23.6	11.7	11.9
Sludge loading ratio, $BOD_5/MLSS$	0.057	0.049	0.045	0.097	0.085	0.073	0.055	0.065	0.100	0.120
Clarifier overflow rate, $m^3/m^2 \cdot d$	7.5	7.0	5.9	6.0	6.7	7.1	9.7	8.1	8.2	8.1
Clarifier HRT, h	7.8	8.3	10.0	9.7	8.7	8.2	6.0	7.2	7.2	7.2
Temperature wastewater, °C	14	16	18	19	20	19	9	10	12	12
Performance Data										
BOD_5 influent, mg/l	204	183	163	208	194	162	176	194	157	186
effluent, mg/l	18	5	5	6	8	11	11	7	9	26
removal, %	91.2	97.3	96.9	97.1	95.9	93.2	93.8	96.4	94.3	86.0
COD influent, mg/l	522	474	293	423	467	391	346	388	351	374
effluent, mg/l	63	57	55	35	58	78	35	48	36	63
removal, %	87.9	88.0	81.2	91.7	87.6	80.1	89.9	87.6	89.7	83.2
Suspended solids, influent, mg/l	228	187	155	234	241	198	149	163	156	209
effluent, mg/l	13	4	4	5	6	7	17	6	6	15
removal, %	94.3	97.9	97.4	97.9	97.5	96.5	88.6	96.3	96.2	92.8
Total nitrogen, influent, mg/l	40.7	42.7	47.1	45.5	46.6	38.0	34.9	38.0	36.8	36.3
effluent, mg/l	9.6	8.3	15.7	6.3	7.0	8.7	7.6	8.6	10.1	13.6
removal, %	76.4	80.6	66.7	86.2	85.0	77.1	78.6	77.4	71.2	57.8

Phosphorus Sources

Sewage effluents carry considerable quantities of phosphorus and nitrogen. When detergents replaced soap as a laundry cleaner, the phosphorus content of sewage increased appreciably. The quantity reached a plateau around 1970 and has decreased somewhat since that time, presumably due to the use of less phosphates as detergent builders.

Human wastes account for 40 to 50 percent of the phosphorus in domestic wastewater. The mean daily human excretion is about 1.5 g per capita (37). Thus, for the 200 million people in the United States, they contribute 110 billion g/year of phosphorus to wastewater. Detergent phosphate builders are estimated to amount to about 2.9 g/capita·d. The total phosphorus concentration in domestic raw wastewater is 5 to 10 mg/l expressed as elemental phosphorus.

Phosphorus Standards

The United States Government is firmly committed to finding solutions for accelerated eutrophication. Although 18 bills relating to the use of phosphorus in detergents have been introduced to the Congress of the United States, none have been enacted. No federal law now exists that bans or restricts the use of phosphates in detergent formulations. Under the Great Lakes Water Quality Agreement of 1972 between Canada and the United States, both governments agreed on the necessity for a 1 mg/l limit for phosphorus for all municipal treatment plants which discharge more than 3785 m^3/d. The two countries took different routes toward achieving the effluent limit. The Canadian government chose to reduce phosphorus discharges both by treatment and by limiting the phosphorus content of detergents. Because of doubts about possible threats to human health of nitrilotriacetic acid (NTA), the Federal government of the United States elected in 1971 to depend on removal of phosphorus in the treatment of sewage. NTA had been developed as a potential substitute for phosphorus in detergents. The International Joint Commission (IJC) at its annual meeting in Windsor, Ontario, in July 1976, stressed the fact that while eutrophication may have slowed somewhat in some places in the Great Lakes, the rate is still more than the natural rate. For this reason, the IJC is now considering whether to lower the effluent limit to 0.1 mg/l and extend it to all municipal plants. They are also considering adoption of a uniform 0.5 percent limit for phosphorus by weight for all detergents manufactured for use in the Great Lakes basin.

The Upper Potomac River Estuary is the other section of the United States in which the amount of phosphorus in wastewater effluents is regulated. The requirements are rather stringent. They call for 96 percent removal of BOD_5, 96 percent removal of phosphorus, and 85 percent removal of total nitrogen.

Several local ordinances ban or restrict the sale of detergents containing phosphorus. Most of the ordinances are similar to the one passed by Chicago. Chicago was the first city in the United States to regulate detergent phosphate content. After February 1, 1971, all laundry detergents sold in the Chicago area had to contain less than 8.7 percent phosphorus and after June 30, 1972, less than 0.5 percent as P. In most localities the ban only applies to laundry detergents and most institutional and industrial cleaners are exempted. That is, detergents used for dairy, beverages, food processing, dishwashers, car washes, hospitals, etc., are exempt.

The trend in the United States is that all practicable measures will be taken to reduce the phosphorus discharged to our surface waters.

Biological Phosphorus Uptake

Biological treatment as practiced today does not effectively remove nitrogen and phosphorus from domestic wastewater. Table 6 shows an estimate of the amount of phosphorus that is being removed by the present methods of treatment. It must be realized that none of the amounts are absolute but represent the going average.

There are two main reasons why conventional biological treatment, as practiced today, does remove nutrients from wastewater. First, modern secondary treatment plants are specifically designed to control the quantity of organic carbon and suspended solids in their final effluent. Secondly, wastewater

TABLE 9. PHOSPHORUS REMOVAL BY WASTEWATER TREATMENT PLANTS

Type	% Removal
Primary sedimentation	5 - 15
Extended aeration (sludge wasting)	8 - 15
Trickling filter	20 - 30
Activated sludge	30 - 50

reaching our wastewater treatment plants today has a nutrient imbalance with respect to the requirements of heterotrophic bacteria for respiration and cell synthesis. This imbalance consists of an excess of nitrogen and phosphorus relative to the amount of carbon. Microorganisms found in biological treatment processes have an average stoichiometric relationship between carbon, nitrogen, and phosphorus of 106 to 16 to 1 atoms in the cell material. Comparing the relative composition of domestic sewage with the mean stoichiometric relation between carbon, nitrogen, and phosphorus in the bacterial organisms, it would follow that the present-day biological treatment system would not be able to assimilate more than 50 percent of the phosphorus into the bacterial cells because of the deficiency of organic carbon. In conventional waste treatment plants the portion of nitrogen and phosphorus that is required for microbial synthesis is retained as activated sludge or trickling filter slime, and the excess nutrients are discharged in the final effluents.

The common practice of primary sedimentation may have the effect of aggravating this imbalance by withholding from aerobic treatment that portion of sewage that is richest in carbonaceous constituents. The advantages and disadvantages of primary settling should be explored.

In most conventional sewage treatment plants, primary and secondary sludges are digested anaerobically to a stable residue and a biologically unstable supernatant liquor. The residue is biologically stable for further processing such as sand-bed drying, and eventual disposal. The supernatant liquor, on the other hand, is grossly polluted and must be further treated. This is normally accomplished by recycle to the influent line of the waste treatment plant. In so doing, large quantities of nutrients are released to the secondary treatment process which further aggravate the already present nutrient-food imbalance in most municipal wastewater. Masselli, et al. (38) listed 413 mg/l NH_4-N and 69 mg/l phosphorus as average values for supernatants from anaerobic digested sludge from 23 sewage treatment plants. See Table 10. If digester supernatant is returned to the treatment plant, the only phosphorus removed will be that associated with the digested sludge.

Since 1950, the phosphorus content of wastewater has increased appreciably, presumably due to the use of phosphates in detergents. This has contributed to the imbalance of nutrients in the raw wastewater and has been reflected in an increase of phosphorus in the final effluent. The present values are considerably higher than the average value of 0.22 mg P/l that Rudolfs (39) found in the trickling filter effluents from 12 American plants during the year 1947. Other surveys, such as those of Johnson (40), Owen (41) and Hume (42), show the same trend.

The most economical and desirable method of increasing phosphorus removal would be to induce a high degree of phosphorus uptake by activated sludge. Storage of complex phosphorus in bacterial cells, when phosphate levels in the medium exist above the basic metabolic requirements, has been reported by several investigators. Anderson (43) has shown that between 1931-1935 and 1951-1955, the phosphorus content of dried activated sludge has increased from a range of 1 to 2 percent in the 1930's to 2 to 4 percent in the 1950's. These values fall well above the 1 percent critical level and indicate that for all practical purposes the organisms are accumulating more phosphorus than the amount required for maximum growth. The quantity stored above the 1 percent required for maximum growth is usually classified as luxury or enhanced uptake. Experiences at the San Antonio plant record phosphorus concentrations in sludge of as much as 7 percent (44).

Levin, at al. (45,46,47,48) have actively studied the enhanced uptake of phosphorus by activated sludge and have developed the PhoStrip process. The configuration of the

TABLE 10. SUMMARY OF SUPERNATANT DATA(38)

Analysis	80% Confidence	Median	Mean
Bicarbonate alkalinity (mg/l as $CaCO_3$)	850 - 2,950	1,580	1,681
5-Day BOD (mg/l)	500 - 4,260	910	1,401
Total phosphorus (mg/l as P)	15 - 124	43	69
Ammonia nitrogen (mg/l)	100 - 710	405	413
pH	6.6 - 8.0	7.3	7.3

process appears in Figure 9. The process is based on the principle that the microorganisms take up phosphorus during the aeration phase of the activated sludge process and then release phosphorus upon being starved of oxygen. A holding tank, called a phosphorus stripping tank, is used to detain (4-12 hrs) the return activated sludge under anaerobic conditions and separate the phosphorus-rich supernatant from the returned activated sludge. The phosphorus is precipitated out of the supernatant, which may contain as much as 80 mg P/l, using lime. Therefore, less chemical is required than if the precipitation is done on the full flow.

The process has been demonstrated at two different sites.

Seneca Falls, New York

The first full-scale installation of the PhoStrip process for phosphorus removal from wastewater was at Seneca Falls, New York. This was a 30-day study that involved modifying an existing conventional activated sludge plant. There had been previous pilot plant testing (47), but none of the pilot tests had ever actually precipitated the phosphorus for final disposition. They had merely demonstrated the achievement of low phosphorus in the final effluent and the production of a high phosphorus supernatant stream. Therefore, none of the problems of diurnal variation in flow and problems of sludge handling have been investigated. The Seneca Falls Paul W. Simson Wastewater Treatment Plant (49) is a modern, dual train, activated sludge treatment plant with a total capacity of 13,247 m^3/day. The plant consists of a bar screen, comminutor, two circular primary clarifiers, two completely mixed aeration basins with mechanical aerators, and two rectangular secondary clarifiers. The waste activated sludge is discharged into the primary clarifier. The major modification consisted of the incorporation of a phosphate "stripping" tank in the return sludge line. The plant had been built in anticipation of receiving wastewater from an adjacent community. The hookup had not been made at the time of the test. The entire flow was processed through only one train of the system. The primary clarifier of the second train was used as the phosphate stripper tank. The sludge is detained long enough to permit it to go anaerobic which induces the sludge organisms to secrete phosphorus. Elutriation of the dissolved phosphorus from the sludge into the supernatant was accomplished by recycling return anoxic sludge back to the stripper tank in the same line conveying secondary sludge to the stripper. The supernatant from the stripper was dosed with lime and discharged back to the primary clarifier for sedimentation of the phosphorus precipitate. Because the supernatant volume was only 14 percent of that of the raw sewage flow, the lime dosage was considerably less than if the whole flow was treated with lime or other chemicals.

The plant performance during the 30-day test period is shown in Table 11. The reported chemical costs based on a lime dosage of 24 mg/l (as CaO) were $2.00 per 3785 m^3. Additional costs would include the stripper tank, the lime storage, mixing and feeding facilities.

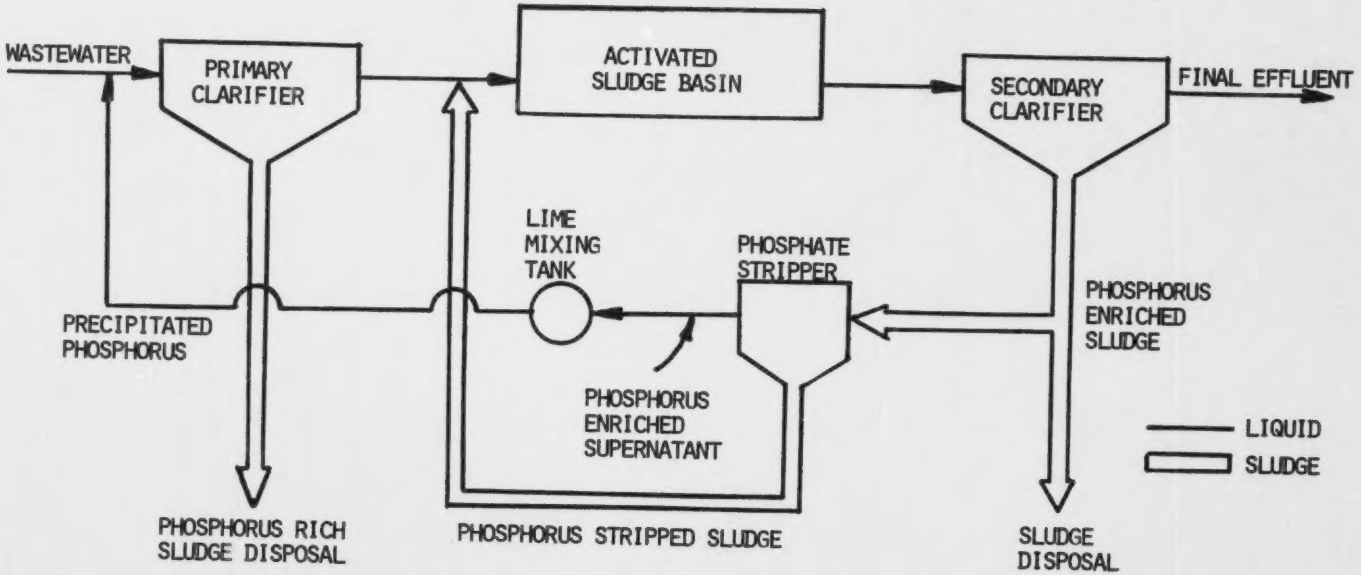

Figure 9. PhoStrip process.

TABLE 11. OPERATING DATA AND RESULTS AT SENECA FALLS(48)

Parameter	Quantity
Plant flow, m^3/d	
Design	6,624
Average	3,406
Influent	
BOD, mg/l	158
P, mg/l	6.3
Average detention times, hours	
Primary clarifier	3.9
Aerator	5.5
Secondary clarifier	3.2
Phosphate stripper	18
Return flows, percent of raw flow	
Sludge to phosphate stripper	24
Sludge to aeration tank	10
Supernatant to primary clarifier	14
Lime dose, mg/l as CaO	24
Suspended solids, mg/l	
Mixed liquor	1,440
Return sludge to stripper	7,840
Return sludge to aerator	15,910
Effluent	
BOD, mg/l	3.5
P, mg/l	0.55
Plant performance	
BOD removal, percent	98
P removal, percent	91

The plant was not equipped to measure sludge flow or total wasting. The question of how much sludge was produced remains to be answered. Another unknown question is whether good phosphorus removal will still be obtained if the plant becomes upset due to toxic wastes.

Reno-Sparks, Nevada

The Reno-Sparks Joint Water Pollution Control Plant was the first plant-scale test of the PhoStrip process operated continuously for an extended period of time. The test program was carried out from June 1974 to June 1975.

The Reno-Sparks Water Pollution Control Plant is a 75,700 m^3/d activated sludge plant that serves the cities of Reno and Sparks, Nevada. The plant consists of three separate modules with common tank walls, separately instrumented and controlled. Each module includes a primary sedimentation tank, an aeration tank, and a secondary sedimentation tank. Oxygenation is provided by diffused air. In the PhoStrip trial operation one of the three primary sedimentation tanks and the grit tank were removed from the normal flow to serve as the phosphorus stripping tank and lime flocculation tank, respectively. The primary effluent was divided equally to the three aeration tanks. This preserved the three identical secondary systems so that one system operated with PhoStrip, and the other two served as controls. Detailed description of the plant and the modification made have been reported by Peirano (50). A schematic of the process used is shown in Figure 10. A summary of the Plant performance is shown in Table 12.

TABLE 12. RENO-SPARKS PHOSTRIP DATA SUMMARY

Parameter	Amount
Wastewater	
Temperature °C	18.7
Raw sewage, m^3/d	22142
Return flows - percentage*	
Aerobic RAS-total	30.6
Aerobic RAS to stripper	22.6
Anaerobic RAS	10.9
Anaerobic RAS recycle to stripper	18.0
Stripping tank supernatant	11.8
Orthophosphate as P	
Primary effluent, mg/l	7.2
Secondary effluent PhoStrip, mg/l	1.1
Secondary effluent control, mg/l	6.0
Stripping tank supernatant, mg/l	51.6
Percentage removal - PhoStrip	85
Percentage removal - control	15.4

*Percentage of wastewater flow.

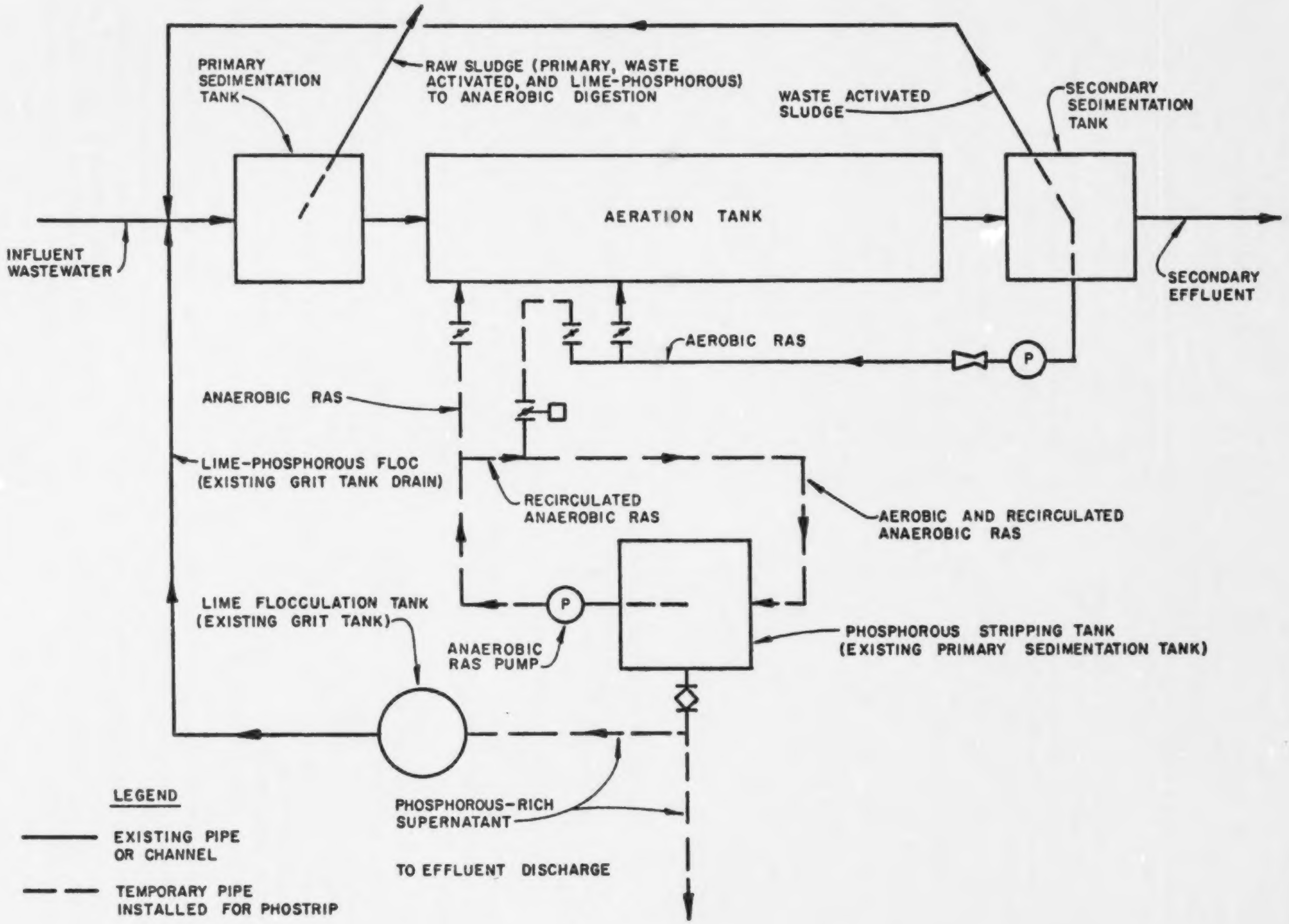
PRIMARY SEDIMENTATION TANK
RAW SLUDGE (PRIMARY, WASTE ACTIVATED, AND LIME-PHOSPHOROUS) TO ANAEROBIC DIGESTION
WASTE ACTIVATED SLUDGE
SECONDARY SEDIMENTATION TANK
INFLUENT WASTEWATER
AERATION TANK
SECONDARY EFFLUENT
AEROBIC RAS
P
ANAEROBIC RAS
LIME-PHOSPHOROUS FLOC (EXISTING GRIT TANK DRAIN)
RECIRCULATED ANAEROBIC RAS
AEROBIC AND RECIRCULATED ANAEROBIC RAS
LIME FLOCCULATION TANK (EXISTING GRIT TANK)
P
ANAEROBIC RAS PUMP
PHOSPHOROUS STRIPPING TANK (EXISTING PRIMARY SEDIMENTATION TANK)
LEGEND
EXISTING PIPE OR CHANNEL
TEMPORARY PIPE INSTALLED FOR PHOSTRIP
PHOSPHOROUS-RICH SUPERNATANT
TO EFFLUENT DISCHARGE

Figure 10. Schematic diagram of PhoStrip process at Reno-Sparks.

In the operation of the plant it was confirmed that the release of phosphorus by the microorganisms is very sensitive to the oxygen present during the anaerobiosis phase. Also it was necessary to keep the sludge blanket level in the secondary sedimentation tank as low as possible to minimize the release of phosphorus into the secondary effluent.

During the course of the Reno-Sparks test program, the worldwide distribution rights for the PhoStrip process were acquired by Union Carbide Corporation. They modified and changed the basic function of the stripper tank. In earlier systems the stripper functioned as a thickener as well as an anaerobic reactor. The size of the stripper, therefore, was determined by the amount of thickening required to maintain the system in equilibrium with respect to solids flow. In the present embodiment the stripper is no longer used to thicken sludge, and the countercurrent leaching of the sludge releases the phosphorus from the sludge. Thus, the reactor volume and the amount of supernatant required to achieve efficient phosphorus removal are significantly decreased.

The U.S. Environmental Protection Agency has funded a grant, "Research Evaluation of PhoStrip Process," submitted by the City of Sparks, Nevada. The City will be the overall project administrators. They will be aided by Union Carbide Corporation and Kennedy Engineers, Inc. The latter firm had demonstrated the earlier PhoStrip technology at this plant.

A full-scale study using the entire flow 75,700 m^3/d is planned. Several operating parameters will be studied to find the best performance. After this has been done, the process will be operated for eight months at defined conditions so that the process variability can be observed in relation to changes in influent and temperature.

A major portion of the effort will be devoted to characterizing the sludge. An engineering report will be produced which will present an evaluation of the maintenance required, reliability of the process, and definition of the operating parameters and economic projection.

SUMMARY

Although basic research is still required to fully understand the chemical and biological transformations that take place, today's technology is adequate to markedly reduce the amounts of nitrogen and phosphorus discharged into receiving waters. The control of phosphorus will, for the present time, be by a chemical system. Engineering improvements and better definition of the essential operating parameters of the biological phosphorus removal process could change this outlook.

Nitrogen removal can be accomplished biologically and when properly exploited has the potential of being efficient, economical and simple. Biological nitrogen removal must be considered an integrated system of nitrification-denitrification. The operation of the first phase is reflected in the output of the other.

A nitrification system can be built to meet today's problems, and the denitrification system can be added in the future if required. Capability for nitrification can be readily added to an existing biological plant and in some cases can be incorporated by minimal changes in operation or design. The amount of additional sludge solids produced is minimal and with biological denitrification, the nitrogen is converted to an ecologically harmless substance. One of the disadvantages of the process is the potential danger of the process being upset by inhibitors. Nitrification is brought about by a rather limited number of species of bacteria with a rather long regeneration time. For this reason, the process tends to be more susceptible to inhibitory materials than are processes for carbonaceous removal. Procedures for monitoring raw waste streams for the presence of such inhibitors are needed. Bioassay methods would probably be more successful than chemical methods because of the wide variety of potentially toxic substances. A major effort should be made on the development of such a procedure, because it may be a cheaper alternative than pretreatment by physical-chemical processes. If extremely low residuals of total nitrogen are to be achieved, information must be developed on the characteristics of the organic nitrogen compounds remaining in a well treated effluent. The concentration of these compounds ranges from a 0.5 to 3 mg/l, and much of it is in the soluble form.

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RESEARCH AND APPLICATION OF WATER RECLAMATION TECHNOLOGY IN SOUTHERN CALIFORNIA

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ABSTRACT

In semi-arid Southern California, water reuse is becoming an increasingly necessary supplement to imported water supplies. This paper describes the Sanitation Districts of Los Angeles County involvement in water reuse and related treatment process research. A brief historical perspective and examination of the potential for water reuse in Southern California is presented. The objective of this paper is to present results of research on water reclamation technology and to describe implementation of the results at several full scale treatment plants. The areas of virus removal, organics removal, nutrient removal, and demineralization have been addressed in the research. Plans exist for conducting further research on health effects of water reuse in a groundwater recharge system that has been operated since 1962.

INTRODUCTION

There are over 11 million people living in the South Coastal hydrologic region of Southern California between the inland deserts and the Pacific Ocean (Figure 1). The limited natural water supply of this semi-arid region, which has an average rainfall of approximately 12 inches per year, is insufficient to support such a population. In order to provide the water essential to the growth of this metropolitan area, three separate aqueducts were constructed to import fresh water over distances of from 240 to 440 miles. Thousands of wells have tapped groundwater supplies to the point where seawater is intruding into freshwater aquifers at many points along the coast. In spite of the obvious need for making maximum possible use of the 1200 million gallons per day of wastewater being produced in the area, only about 60 million gallons per day is intentionally reused (1). Most of the remainder is discharged to the ocean.

The Sanitation Districts of Los Angeles County provide regional sewerage services for almost 4 million people in 73 cities within the South Coastal hydrologic region (Figure 1). The Districts operate 11 treatment plants with a combined flow of 430 million gallons per day of which 26 million gallons per day is reused. The flow from 10 of the treatment plants is reused in various modes as delineated in Table I. The 26 million gallons per day reused at these locations constitutes half the total reuse in the South Coastal region. Improvements in treatment facilities and research to develop new low cost technology and resolve public health issues should lead to expanded reuse programs.

Over the last 30 years, public interest in water reuse projects has been directly related to the availability of natural supplies. There was little interest and no action in the decade of the 1950's because the aqueduct delivering imported water from the Colorado River had substantial excess capacity. In 1959, legal decisions reducing rights to Colorado River water, the uncertainty of the California Water Project, and declining groundwater elevations stimulated public interest and support for water reuse. Spurred by this concern, agreements were arranged to fund and operate the 10 million gallon per day Whittier Narrows Water Reclamation Plant. Since it commenced operation in 1962, over 220,000 acre feet of reclaimed water has been spread to recharge groundwater basins in Los Angeles County at a cost to water agencies of over $4 million.

Figure 1

LOCATION MAP

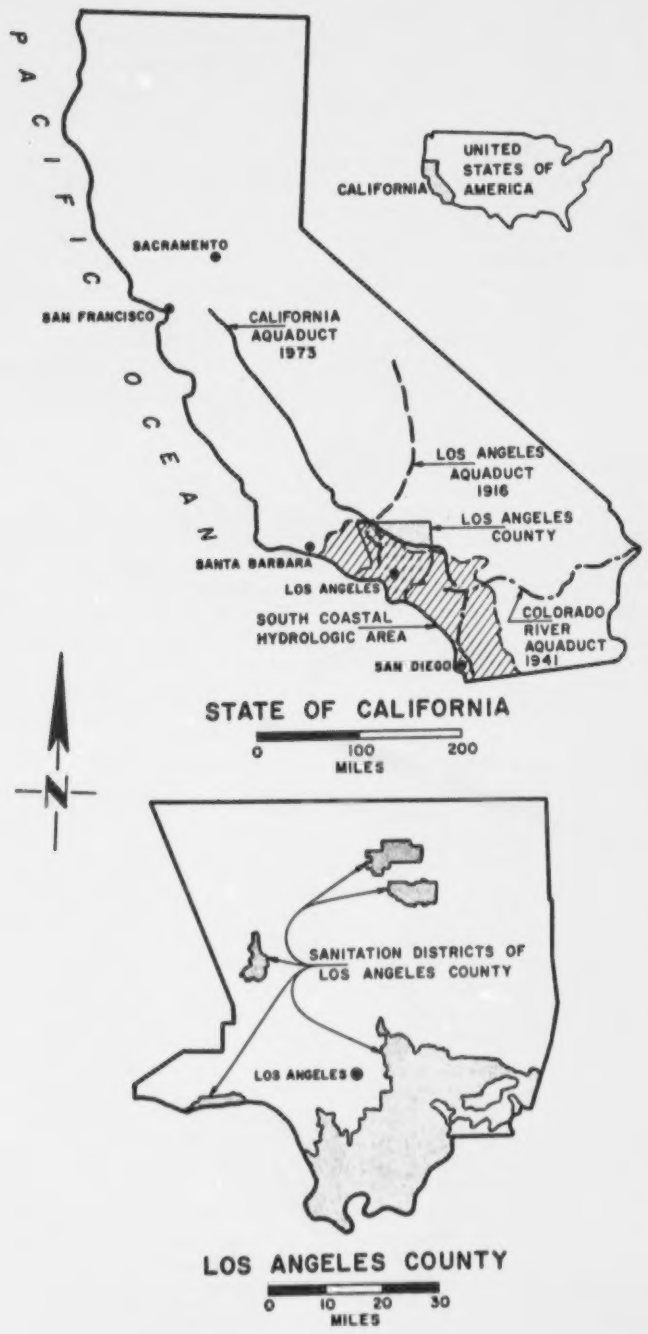


TABLE I. WATER REUSE FACILITIES OF THE SANITATION DISTRICTS OF LOS ANGELES COUNTY

Reclamation Plant	Avg. Flow (mgd)	Average Quantity Reused (mgd)	Types of Reuse	TDS (mg/l)	Hardness (mg/l)
Whittier Narrows	14	14	Recharge	510	180
San Jose Creek	30	10	Recharge	680	230
Pomona	8	1 7	Irrigation Incidental Recharge	560	210
Los Coyotes	20	0	No present reuse	830	240
Long Beach	7	0	No present reuse	630	160
Lancaster	4	0.3	Recreation Lakes and Park Irrigation	580	90
Palmdale	2	1	Alfalfa Irrigation	470	80
La Canada	0.1	0.1	Golf Course Irrigation	700	190
Saugus	3	3	Incidental Recharge	900	330
Valencia	2.5	2.5	Incidental Recharge	920	290
Total Flow	91 (mgd)				
Intentional Reuse		26 (mgd)			
Incidental Reuse		13 (mgd)			
Total Reuse		39 (mgd)			

Based on the success of the Whittier Narrows Plant, the Sanitation Districts developed a master plan for construction of water reclamation plants to meet the need for expanded wastewater treatment capacity. However, the implementation of the California Water Plan to provide additional imported water to the area reduced the interest of water agencies to the point where relatively little of the reclaimed water potential has been used to date.

With the construction of the California Aqueduct in 1973, interest in reusing water again declined. Water agencies were more concerned with maximizing purchases of imported water to minimize unit costs and placed great emphasis on the lower dissolved solids concentration of water imported via the aqueduct from Northern California.

In the last few years, Californians have become conscious of the need to reuse water. Recently, the State Water Resources Control Board, which regulates wastewater facilities, issued a policy designed to require that water reuse be implemented wherever feasible (2). This policy resulted in part from a desire to avoid placing dams on the few rivers in Northern California not already being utilized for water supplies and, in part, from the recognition that California was experiencing a severe drought.

As the drought has continued into its second year with record low rainfalls of less than 30 percent of normal, it appears timely to expand the reuse of water with the general support of a water-conscious population.

POTENTIAL AND PROBLEMS ASSOCIATED WITH EXPANDING REUSE

In Southern California, only about half of the water delivered to urban areas reaches the sewerage system. Only about 50 percent of the wastewater in the sewers has dissolved solids concentrations under 1000 mg/l and can be reclaimed and reused without expensive desalination processes. The maximum practical reuse potential is therefore about 600 million gallons per day, or about ten times the amount actually being intentionally reused. In the Sanitation Districts' system, it is estimated that less than 200 million gallons per day of a projected 1995 flow of 500 million gallons per day is so located as to permit its reuse. The major potential for reuse is groundwater basin recharge because of the availability of spreading basins in close proximity to treatment facilities. The advantages of groundwater basin recharge are that the soil mantle provides tertiary treatment and protection against the transmission of disease, and the underground aquifer provides both a distribution system for the water and a method of losing its identity by mingling with other natural waters.

Recently, the California State Health Department has expressed concern over the hypothesized relationship between low concentrations of stable organics reaching groundwater supplies and their effects on human health. Until research efforts and epidemiological studies can alleviate this concern, the Health Department stands opposed to expanding spreading operations, except in selected cases to permit further study. Recently, the Health Department has proposed treating secondary effluent with activated carbon before spreading it on the ground in order to reduce organic residues. These proposals are still under discussion.

It is estimated that about 20 million gallons per day could be utilized for the irrigation of parks, golf courses, and agricultural lands. Irrigation is a desirable method of using reclaimed water because the nutrients, nitrogen and phosphorus, are not only utilized by most crops but reduce the need for supplying fertilizer. Unfortunately, the agricultural developments of Southern California are generally quite distant from the metropolitan areas producing large quantities of usable reclaimed water. Within metropolitan areas, there will be some use for irrigation of large parks, cemeteries, and landscaping along freeways. Health Department requirements include having a well-oxidized and stable secondary effluent, filtered and disinfected to a coliform MPN of 23/100 ml for this purpose. Reclaimed water used for irrigation of fodder crops and other crops that are not ingested by man do not have to receive treatment beyond primary to meet Health Department rerequirements. However, secondary treatment is now mandatory throughout the nation as a uniform minimum standard.

There is a limited potential for the industrial use of reclaimed water

The most obvious demand is for makeup water to cooling towers for heat dissipation. In Burbank near Los Angeles, about 2 million gallons per day of secondary effluent is used in cooling towers operated by the city, and it is estimated that 25 million gallons per day could be used for that purpose in the Districts' system. The use of reclaimed water for industrial process purposes is more specialized, but the Sanitation Districts anticipate that two paper-producing plants near Pomona may soon be able to meet their high water demands with carbon-filtered water. The problems associated with industrial reuse are related to overcoming psychological objections, the need to provide dual water systems and health safeguards, and the problems associated with chemical constituents, such as higher dissolved solids, slime growths related to nutrients, or hardness.

The use of reclaimed water to supply lakes used for recreational purposes has been successfully practiced at two locations in Southern California, Santee near San Diego and Lancaster at the northern edge of Los Angeles County. Where natural soils are not available to assist in the treatment, the production of water suitable for recreational use will be very expensive. There are unique situations, however, where the scarcity of water and the need to provide high levels of treatment prior to disposal of wastewater will make such projects acceptable.

In summary, it is anticipated that, over the next 25 years, about half of the wastewater produced in Southern California will be reused, primarily for agricultural and groundwater recharge purposes. Utilization of the reclaimed water will be dependent upon the scarcity of water and the resolution of health issues associated with minute concentrations of residual organics, since such use may involve blending reclaimed water with other supplies to permit their use for all purposes.

DEVELOPMENT OF WATER RECLAMATION TECHNOLOGY

Groundwater Recharge

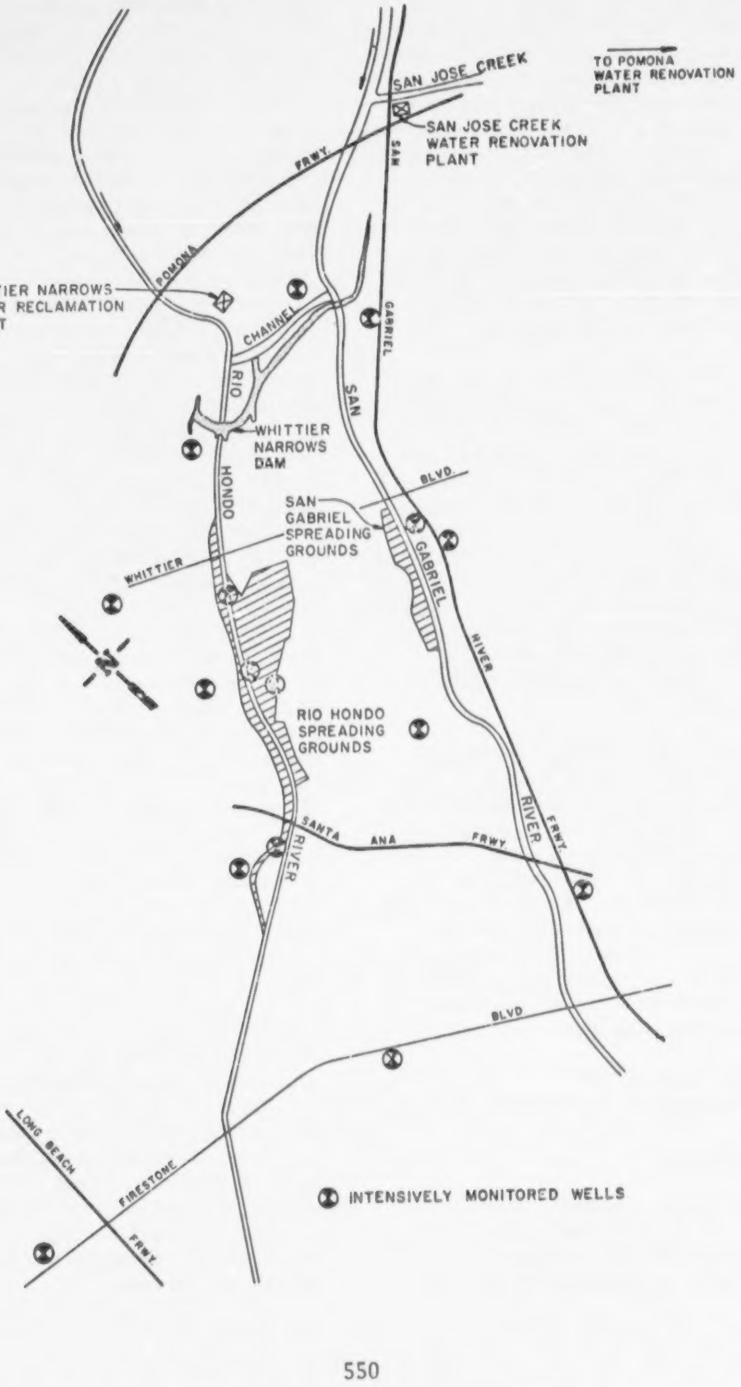
Intentional water reuse by groundwater recharge with reclaimed wastewater has been practiced in Los Angeles County since 1962. Three Sanitation Districts' activated sludge treatment plants produce secondary effluent, which is used for groundwater recharge. The secondary effluent is disinfected by chlorination and discharged to flood control channels that convey the water to spreading grounds where it is allowed to percolate into groundwater aquifers used for domestic supplies. The total quantity of reclaimed water that is percolated in this system is presently 25 million gallons per day. The spreading grounds, which are located along the Rio Hondo and San Gabriel Rivers, as shown in Figure 2, have total areas available for percolation of 455 and 235 acres respectively. The spreading grounds are subdivided into numerous individual basins ranging in size from 4 to 20 acres. Batteries of basins are rotated through an 18-day cycle consisting of filling to a depth of 4 feet for six days, draining for six days, and drying for six days. This maintains aerobic soil conditions and prevents insect infestations. On this rotating basis, the capacity of the basins for recharge is 200 million gallons per day. During storms, in order to maximize conservation of storm water, all basins are flooded, in which case the short term capacity for groundwater recharge is approximately 600 million gallons per day.

Each year, approximately 180,000 acre-feet of water must be percolated into the groundwater basins to replace pumped water. Of this, approximately 60,000 acre-feet consists of captured runoff, subsurface lateral flow into the basin, and water that has risen to the surface in an adjacent groundwater basin. The remaining 120,000 acre-feet consists of reclaimed wastewater and imported surface supplies. The total reclaimed water input to the groundwater basin is presently 27,000 acre-feet per year. Since 1962, the total quantity of reclaimed water recharged is approximately 300,000 acre-feet.

The present practice of groundwater recharge includes dilution of reclaimed water with imported surface supplies in a 2:1 ratio of imported to reclaimed water. This dilution criterion was intended to consistently produce groundwater with less than 10 mg/l of nitrate nitrogen. When subsurface inflow and storm water are considered, the percentage of reclaimed water input to the groundwater basin is approximately 15 percent.

FIGURE 2

LOCATION MAP
SPREADING GROUNDS AND TREATMENT PLANTS


TO POMONA WATER RENOVATION PLANT
SAN JOSE CREEK
SAN JOSE CREEK WATER RENOVATION PLANT
POMONA FRWY.
WHITTIER NARROWS WATER RECLAMATION PLANT
CHANNEL
RIO HONDO
WHITTIER NARROWS DAM
SAN GABRIEL
SAN GABRIEL RIVER
SAN GABRIEL SPREADING GROUNDS
BLVD.
WHITTIER
N
RIO HONDO SPREADING GROUNDS
SANTA ANA FRWY
RIVER
SAN GABRIEL RIVER FRWY.
FIRESTONE BLVD
LONG BEACH FRWY.
INTENSIVELY MONITORED WELLS

Domestic water supply pumping wells are located at various distances from the spreading grounds. Some of the wells are located on the borders of the spreading grounds. The water pumped from wells in the general vicinity of the spreading grounds makes up about 40 percent of the domestic supply served to more than 1.5 million people who reside in south-central Los Angeles County.

Monitoring of wells surrounding the spreading grounds was begun by the Los Angeles County Flood Control District in the mid-1950's. Subsequently, an intensive monitoring program was established in 1972 as a joint effort by the Los Angeles County Sanitation Districts, the Flood Control District, and the Central and West Basin Water Replenishment District. The purpose of the intensive monitoring program is to characterize the quality of all waters used for groundwater replenishment and the quality of water withdrawn from the wells. Intensively monitored wells are located at distances varying from zero to four miles from the nearest spreading grounds. Control wells are located hydrologically upstream of the spreading grounds. Samples of replenishment waters and wellwaters are analyzed for constituents, such as TDS, hardness, nitrates, trace metals, COD, chlorinated hydrocarbons, radioactivity, and bacteria. Virus samples are collected at the three treatment plants and at one well. Carbon chloroform extract samples have been collected at the same well and at two of the three treatment plants. An outline summary of the intensive monitoring effort is contained in Table II.

Results of reclaimed water quality monitoring are summarized in Table III. As indicated, the three treatment plants produce comparatively high-quality secondary effluent with suspended solids ranging from 4 to 7 mg/l, COD ranging from 31 to 43 mg/l, and total nitrogen ranging from 13 to 19 mg/l. Wellwater quality is summarized in Table IV in terms of chemical oxygen demand and nitrate. The range of average chemical oxygen demand concentrations in 16 intensively monitored wells is 1.2 to 4.8 mg/l. As indicated in Table IV, these wells are located at distances varying from zero to 3.8 miles from the spreading grounds, and the perforations in the casings of these wells range from as little as 40 feet to as much as 530 feet. The groundwater COD concentrations are fairly low compared to those found in domestic supplies in Southern California that originate from surface waters. For example, the average chemical oxygen demand in Colorado River aqueduct water is approximatley 10 mg/l and the COD in the Los Angeles aqueduct water is approximately 4 to 6 mg/l. With respect to COD, no trend with distance from spreading ground or depth of perforations can be recognized.

Nitrate concentrations in wells in the vicinity of the spreading grounds range from 0.1 to 4.3 mg/l, which is below the established health criterion of 10 mg/l as nitrogen in the United States. A trend in nitrate concentration related to proximity to the spreading grounds seems to exist. Wells close to the spreading grounds and with shallow perforations are generally higher in nitrate with concentrations ranging from 2.3 to 4.3 mg/l as nitrogen. Control wells upstream of the spreading grounds are fairly low, ranging from 0.1 to 0.6 mg/l, and wells that are farther away from the spreading grounds that would probably be minimally influenced have nitrate concentrations ranging from 0.4 to 3.3 mg/l.

With respect to virus and carbon chloroform extract monitoring, limited data have been developed. Viruses have been isolated in chlorinated secondary effluents from two of the three treatment plants. However, viruses have not been detected in samples collected at a shallow well located within the spreading grounds. Carbon chloroform extract samples collected at the shallow well indicated that the CCE in groundwater is consistently below the drinking water limit of 0.7 mg/l. Because this well provides the worst case in terms of protential contamination, it is felt that the percolation system provides excellent removal of viruses and protection against organic contamination of the surrounding well system.

Virus Removal and Tertiary Treatment Systems

In addition to the indirect potable reuse described in the preceding section, water is reused in Southern California for recreational purposes. Five Districts' treatment plants discharge chlorinated secondary effluent into the Rio Hondo and San Gabriel Rivers. The present flow from these five plants is approximately

TABLE II

CONSTITUENTS MONITORED IN RECLAIMED WATER, IMPORTED WATER, AND GROUNDWATER (16 WELLS)

Constituents	Reclaimed Water	Imported Water	Groundwater
Mineral Ions	M	SA	Q
Nitrogen Compounds	D	SA	Q
Chemical Oxygen Demand	D	SA	Q
Metals and Pesticides	M	SA	A
Coliform Bacteria	D	-	M
Virus	M	-	M

Notes: D = daily
M = monthly
Q = quarterly
SA = semi-annually
A = annually

TABLE III

SUMMARY OF SECONDARY EFFLUENT QUALITY

Constituents	Pomona	San Jose Creek	Whittier Narrows
Turbidity, FTU	3.8	2.2	3.5
Suspended Solids, mg/l	6	4	7
Total COD, mg/l	43	31	36
Dissolved COD, mg/l	35	25	27
Total Nitrogen, mg/l N	17.7	13.7	18.6
Total Phosphorus, mg/l PO_4	22.8	22.1	18.9
Total Dissolved Solids, mg/l	559	680	509
Total Coliform, MPN/100 ml	<2	<3	144

TABLE IV

SUMMARY OF WELL WATER QUALITY

1. Wells close to the spreading grounds and with shallow perforations

No. of Wells	Distance from Spreading grounds, miles	Depth to First Perforation, ft	Avg. COD, mg/l	Avg. NO_3^-, mg/l N
7	0-0.8	40-300	3.1	3.2

2. Wells moderately close to the spreading grounds with deeper perforations

No. of Wells	Distance from spreading grounds, miles	Depth to First Perforation, ft.	Avg. COD, mg/l	Avg. NO_3^-, mg/l N
5	0.8-3.8	300-500	3.5	1.7

3. Wells upstream from spreading grounds

No. of Wells	Distance from spreading grounds, miles	Depth to First Perforation, ft.	Avg. COD, mg/l	Avg. NO_3^-, mg/l N
4	1.0-2.0	145-530	3.1	0.2

80 million gallons per day. During much of the year, the rivers contain undiluted secondary effluent and, because of the warm climate, swimmers frequent the rivers. Because of this human contact, discharge to the rivers must comply with the criteria specified in Title 22 of the California Administrative Code for wastewater discharged into unrestricted recreational impoundments. The intent of these criteria is to protect swimmers from water-assocciated viral illness. Viruses normally occur in low concentrations in treated wastewater and their assay demands special expertise; hence, a specific limit on virus concentration is not required by the criteria. Instead, the criteria specify a tertiary treatment system consisting of coagulation-sedimentation-filtration-disinfection, and specify that the effluent coliform concentration must be less than or equal to 2.2 MPN/100 ml (7-day median). Additionally, the criteria state that alternative methods of treatment may be utilized subject to evaluation on a case-by-case basis.

Because the specified system is costly with respect to both capital and operational expenses, an incentive existed for evaluating alternative treatment systems. At the Sanitation Districts' Pomona Research facility, several tertiary treatment systems were investigated on a pilot scale to compare their virus removal efficiency. The system described in Title 22 was used as the baseline in the comparison. The tertiary treatment systems evaluated included the following:

System A: Alum coagulation, flocculation, sedimentation, filtration, disinfection (Title 22 system).

System B: Low dose alum coagulation, filtration, disinfection.

System C: Carbon adsorption, disinfection, carbon adsorption.

System D: Low dose alum coagulation, filtration, disinfection (nitrified effluent feed).

Both ozone and chlorine were tested as the disinfectant.

Objectives

The study was divided into two phases dealing with seeded and naturally occurring viruses. The objectives of the study were to:

1. Rank the treatment systems in terms of virus removal efficiency and reliability by conducting seeding experiments.

2. Establish, if possible, functional relationships between water quality parameters and virus removal.

3. Determine types and concentrations of naturally occurring viruses in secondary effluent and tertiary effluents.

4. Derive cost estimates for systems performing equivalently to System A in terms of seed virus removal.

Methods

Treatment Systems - There are two activated sludge treatment plants at the Pomona site. Non-nitrified secondary effluent, which fed Systems A, B, and C, was provided by a 10-mgd activated sludge system. Nitrified secondary effluent, used as feedwater in System D, was provided by a 1.5-mgd, two-sludge nitrification system. The pilot tertiary treatment systems were of comparatively large scale ranging from 25 to 50 gpm. Major operating parameters of the pilot systems are summarized in the schematic diagram of each system contained in Figure 3.

Virus Sampling Procedure - The sampling procedure, capable of detecting low concentrations of virus in large volumes of wastewater, consisted of three phases.

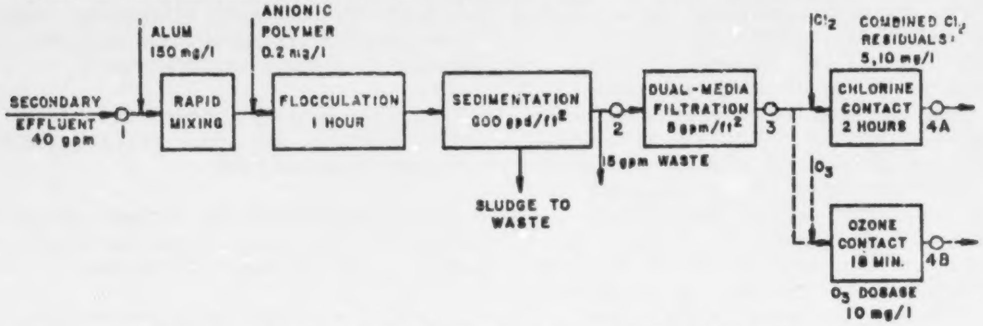
1. Membrane adsorption
2. Chemical precipitation
3. Soluble alginate filtration

A series of adsorbent filters was used to concentrate viruses contained in water samples ranging in volume from 20 to 100 gallons. The viruses were than eluted from these filters and further concentrated using the chemical precipitation and alginate filtration techniques. The detection limit of the method is 20 PFU/100 gallons of wastewater.

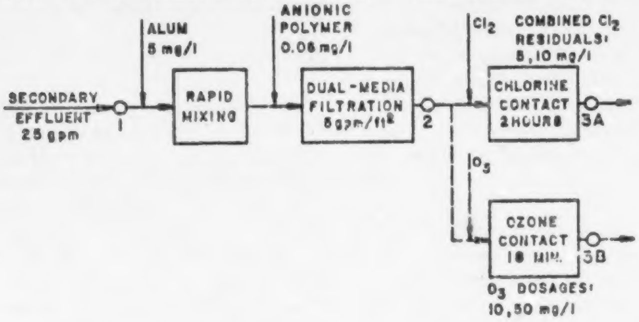
Assays of samples collected during poliovirus seeding studies at Pomona were performed by the plaque technique using primary African Green monkey kidney cells

FIGURE 3: PILOT TERTIARY TREATMENT SYSTEMS

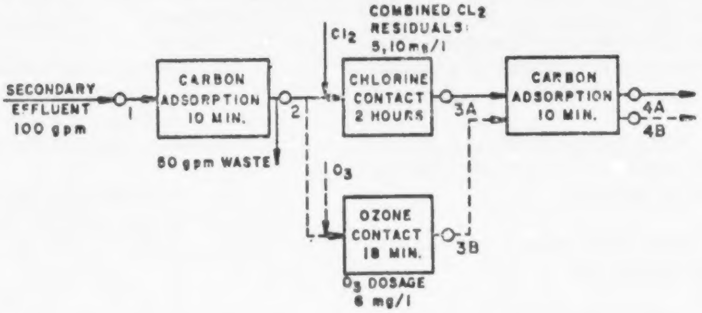
SYSTEM A: COAGULATION, SEDIMENTATION, FILTRATION AND DISINFECTION



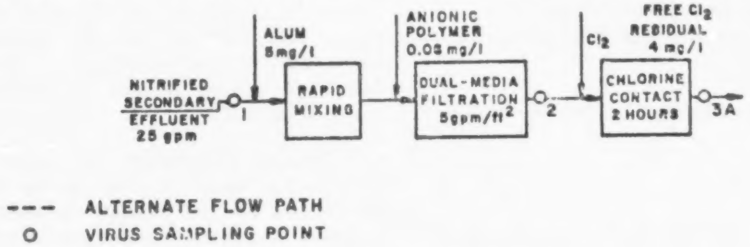
SYSTEM B: COAGULATION, FILTRATION AND DISINFECTION



SYSTEM C: TWO-STAGE CARBON ADSORPTION AND DISINFECTION



SYSTEM D: NITRIFICATION, FILTRATION AND DISINFECTION



as the host cell line. During natural virus experiments, samples were assayed by inoculation into screw cap tubes of four cell lines. The cell lines used were primary African Green monkey kidney, primary Rhesus monkey kidney, WI-38, and HEp-2. Those samples exhibiting cytopathogenic effects (CPE) on any of the four cell lines were quantitated by plaquing on primary African Green monkey kidney cells. Identification was accomplished by use of specific antisera.

Conduct of Experiments - To allow comparison of virus removal efficiencies by the treatment systems, seeding experiments were conducted. The seed virus utilized was polio type 1 (Chat). The experiments were planned such that a detectable virus concentration would be found in the final effluent from each of the systems so that virus removal efficiency could be calculated. The seed virus concentration in the influent of each system was approximately 5 x 10^7 PFU/100 gal. Virus samples were collected during the seeding studies downstream of each unit process within the systems. Virus sample volumes were 20 gallons for secondary effluent, 50 gallons for settled and filtered effluents, and 100 gallons for post-disinfection effluents.

To minimize uncertainties associated with proper sampling time, a concentrated virus solution was injected continuously into the secondary effluent supply of each system over the theoretical hydraulic detention time of the system. Prior to the virus experiments, tracer experiments were conducted to determine the proper sampling times at downstream locations.

In addition to the virus seeding experiments, a separate series of natural virus experiments was conducted. As in the seeding experiments, a secondary effluent sample volume was 20 gallons and the final effluent sample volume was 100 gallons. It was hoped that information developed on types and concentrations of viruses in secondary and tertiary effluents would be valuable in beginning to assess the health risks associated with water reuse. The natural virus experiments were conducted in the fall because peak virus concentrations were expected in this period. All seeded and natural virus samples were accompanied by samples for water quality analyses.

Results

Seedings Experiments - Seeding experiments in systems employing combined residual chlorination were conducted at two chlorine residual levels. One set of experiments utilized an average chlorine residual of approximately 5 mg/l (combined) in Systems A, B, and C. During a second set of experiments, the average chlorine residual was approximately 10 mg/l. In subsequent discussions, the 5 mg/l and 10 mg/l residuals will be referred to as low and high chlorine residuals, respectively. Experiments with System D were conducted at a free chlorine residual of 4 mg/l. Chlorine demand varied from 4 to 6 mg/l; hence, dosages of approximately 10 and 15 mg/l were used in Systems A, B, and C. In System D, a 10 mg/l dosage was required. Average water quality produced by the systems is summarized in Table V.

Results of seeding experiments using combined residual chlorination are summarized in Table VI. At average chlorine residuals of approximately 10 mg/l, Systems A, B, and C all removed approximately 5.2 logs of seeded polio virus (99.9994 percent) despite slight differences in predisinfection water quality. System D removed approximately 4.9 logs (99.999 percent) using 4 mg/l free chlorine residual. At combined residuals averaging 5 mg/l, differences in virus removal capability of the systems were apparent. System A removed 5.1 logs (99.9992 percent) compared to 4.7 logs (99.998 percent) for System B. Virus removal by System C did not significantly decrease at the lower chlorine residual and remained comparable to System A.

The conclusion drawn from these results, which was based on statistical analysis of the data, was that System B could perform equivalently to the required system provided high chlorine residuals were employed. System C performed equivalently at both high and low chlorine residuals, and System D provided no benefit over System B because the cost of nitrification outweighed the savings in chlorine dosage.

Overall virus removals when ozone was used ranged from 5.2 to 5.4 logs, which is slightly greater removal than that obtained in chlorination with average residuals

TABLE V

AVERAGE PREDISINFECTION WATER QUALITY PRODUCED BY PILOT ADVANCED WASTEWATER TREATMENT SYSTEMS

CONSTITUENT	TREATMENT SYSTEMS			
	A	B	C	D
Suspended Solids, mg/1	1.3	2.7	2.2	2.8
Turbidity, FTU	0.7	1.2	1.5	1.5
Color	19	27	11	28
Total COD, mg/1	20	24	17	25
pH	6.9	7.4	7.5	7.3
NH_3-N, mg/1	21	22	20	0.1
NO_2-N, mg/1	0.2	0.4	0.8	0.1
NO_3-N, mg/1	8.2	9.5	6.0	12
TDS, mg/1	590	558	520	580

TABLE VI

VIRUS REMOVAL IN SYSTEMS EMPLOYING CHLORINATION

SYSTEM	AVERAGE CHLORINE RESIDUAL, mg/l	GEOMETRIC MEAN VIRUS REMOVAL %	Logs
A	4.9	99.9992	5.1
	10.0	99.9994	5.2
B	5.2	99.998	4.7
	10.1	99.9994	5.2
C	5.4	99.9992	5.1
	11.7	99.9994	5.2
D	3.9 (free)	99.999	4.9

of 10 mg/l. An ozone dose of 10 mg/l was used in System A, doses of 10 and 50 mg/l were used in System B, and a dose of 6 mg/l was used in System C. Experiments on System D were limited.

During the virus seeding experiments, final effluent coliform samples were collected at the same time as final effluent virus samples. High chlorine residuals produced a median coliform concentration of less than or equal to 2 MPN/100 ml as did free residual chlorination in System D. The coliform data reflect the same trend as the virus data. At low chlorine residuals, differences in the median final effluent coliform concentrations reflected slight differences in predisinfection water quality among the systems. At high chlorine residuals, little difference existed among the systems in terms of production of effluent meeting the coliform standard. Data from the ozonation experiments indicate that relatively high coliform concentrations in final effluent may accompany excellent poliovirus removal by ozonation.

Virus removal in unit processes used for preparing the wastewater for disinfection was determined. Coagulation with a low dose of alum followed by filtration through an inert media filter (System B) produced an average virus removal of 1.0 log (90 percent). Single-stage carbon adsorption with a ten-minute contact produced approximately the same degree of virus removal averaging 1.1 logs. Coagulation and filtration of nitrified effluent (System D) also yielded an average removal of 1.0 log. Coagulation, sedimentation, and filtration using 150 mg/l of alum, as practiced in System A, produced 2.3 logs (99.5 percent) virus removal. As indicated by comparison of these removals with those reported above for entire systems, the majority of virus removal occurs in the disinfection step. The function of the tertiary treatment systems is to produce a well-clarified effluent with high reliability so that efficient disinfection can occur.

Natural Virus Experiments - In 60 samples of unchlorinated secondary effluent, naturally occurring viruses were detected in 27. Prior to beginning the natural virus experiments, it was expected that viruses would be isolated in unchlorinated secondary effluent samples with high frequency and that virus concentrations in the effluent would range typically from 2,000 to 5,000 PFU/100 gallons. Reports of natural virus sampling in unchlorinated secondary effluent are sparse. In studies conducted in the United States in recent years (3, 4) virus concentrations detected in unchlorinated secondary effluent ranged from 300 to 15,000 PFU/100 gallons. Concentrations detected in unchlorinated secondary effluents in foreign countries yielded much higher concentrations from 10^6 to 10^7 PFU/100 gallons (5, 6).

Concentrations in unchlorinated secondary effluent in the Pomona experiments ranged from 165 to 51,700 PFU/100 gallons and, in several positive samples, plaques were not detected when cytopathogenic effects had been observed in the tube assay procedure, which is more sensitive. The geometric mean in the secondary effluent samples that yielded plaques was 1880 PFU/100 gallons. Virus types detected in secondary effluent included polio 1, 2, 3, ECHO 1, REO, coxsackie B2, and adeno.

Natural virus sampling in tertiary effluents resulted in detection of viruses in nine of 60 experiments. Virus types recovered in tertiary effluents included REO, ECHO 1, polio 1, 2, and 3. Viruses were recovered in ozonated effluents from Systems A, B, and C, and in chlorinated effluents from Systems B and D. Virus concentrations detected in the tertiary effluents ranged from 65 to 420 PFU/100 gallons. Results of virus sampling in secondary and tertiary effluents are summarized in Table VII.

Because naturally occurring virus concentrations were highly variable, the results did not lend themselves to statistical analysis. The nine positive tertiary effluent samples would not have been expected based on the detected secondary effluent concentrations and the seeding results which indicated 5 logs removal. Two factors that may partially explain the unexpected results are the greater resistance of natural viruses and the lower natural virus concentrations in the influent to each system.

Correlations with Water Quality Parameters - One of the primary objectives of the study was to determine whether water quality could be correlated with virus removal. Statistical analyses (linear regression) were performed to investigate correlations between predisinfection water

TABLE VII

SUMMARY OF NATURAL VIRUS RESULTS

TYPE OF EFFLUENT	NO. OF SAMPLES	NO. OF POSITIVE SAMPLES	RANGE OF CONCENTRATIONS DETECTED, PFU/100 gal	TYPES DETECTED
Unchlorinated Secondary	60	27	165-51,700	Polio 1,2,3, Coxsackie B2, REO, ECHO 1, Adeno
A-Cl_2	10	0	None	None
A-O_3	7	3	<15-71	Polio 1,3, ECHO 1, REO
B-Cl_2	11	1	140	ECHO 1
B-O_3	7	2	<15-65	Polio 1, ECHO 1
C-Cl_2	7	0	None	None
C-O_3	7	1	305	Polio 2
D-Cl_2	7	2	<15-420	Polio 1
D-O_3	5	0	None	None

quality parameters and effluent virus concentration. Additionally, the relationship between effluent coliform and effluent virus concentration was sought. Using virus data pooled from all systems employing combined residual chlorination, it was found that chlorine residual and predisinfection virus concentration affect effluent virus concentration most strongly. In ozonation experiments, linear regressional analyses indicated that predisinfection COD, virus concentration, color, and turbidity influenced the effluent virus concentration. The relationships derived illustrate trends but do not yield a precise model that could be used for operational control. The analyses provide a basis for further research that could yield an operational control model. Detailed results of the statistical work are contained in a separate report (7).

An attempt was made to determine if the coliform standard established for recreational waters (median MPN ≤ 2.2/100 ml) could be used as an index of virus inactivation. A figure was constructed by grouping the virus removal data from chlorination experiments into half-log intervals and finding the percent of the time that coliform concentrations corresponding to these virus removal data were equal to or less than 2.2 MPN/100 ml. The figure indicated that a median coliform concentration of less than 2.2 MPN/100 ml limit was achieved in those experiments where the poliovirus removal was approximately 5 logs. This degree of virus removal was attained by System A, the system required by the regulations. From the experiments conducted at Pomona with well-clarified tertiary effluents, it appears that satisfaction of the stringent 2.2 MPN/100 ml coliform standard coincides with achievement of adequate virus removal, which is defined by the virus removal attained in System A.

Data generated in the virus study indicate a smaller than expected difference in virus inactivation between combined and free residual chlorination. Previous experiments indicated that free residual chlorination would provide much higher efficiency of virus inactivation than combined residual chlorination. However, several of these studies were conducted in demand-free distilled water and in several others chlorine and ammonia were pre-combined before addition of viruses. Other experiments indicated that efficient mixing upon introduction of chlorine into wastewater improved virus inactivation significantly. It was hypothesized that free residual chlorine existed for a short period of time upon efficient mixing and that substantial virus inactivation occurred nearly instantaneously upon mixing of chlorine with wastewater (8). In the experiments conducted at Pomona, rather high input energies were utilized in chlorine mixing and approximate Reynolds numbers achieved were on the order of 180,000, and it is possible that this greatly enhanced the virus inactivation in combined residual chlorination.

Experiments were conducted in which seeded secondary effluent was chlorinated without prior filtration. The results of these experiments indicated that average virus inactivation was approximately 4.3-4.5 logs and that the results were variable. These results reinforced the need for some mode of filtration prior to disinfection to ensure reliable disinfection performance.

Based on an analysis of variance of effluent virus concentrations, a list of treatment systems that performed equivalently in terms of virus removal was generated. However, further analysis using probability plots indicated that several of those systems listed performed unreliably. A final list of systems found comparable to the State Health Department standard system (System A) was generated and is listed as follows:

System A (5 mg/l combined chlorine residual)
System B (10 mg/l combined chlorine residual)
System C (5 mg/l combined chlorine residual)
System D (5 mg/l free chlorine residual)

System A (10 mg/l ozone dosage)
System B (50 mg/l ozone dosage)

Cost Estimate

Systems that performed equivalently with respect to virus removal and with equal reliability were subjected to cost analyses. The direct filtration system (System B) with chlorination is the lowest cost system. The total capital and operational cost of System B is estimated to be 13.7¢/1000 gallons treated compared to

21.5¢/1000 gallons for System A with chlorine, 17.2¢/1000 gallons for System C with chlorine, and 19.9¢/1000 gallons for System D. The systems employing ozonation were more costly. Estimated costs for the systems listed above are contained in Table VIII.

Conclusion

Based on the results of these virus experiments, it was concluded that direct filtration (System B) or carbon adsorption (System C) tertiary treatment systems are more cost effective than the required system (System A) for treatment of secondary effluent that is to be disinfected and discharged into unrestricted recreational impoundments. These alternate systems reliably produce well-clarified effluent suitable for efficient disinfection. After such treatment, combined residual chlorination can be employed to produce an overall virus removal of 5 logs, which was achieved in the required system used as a baseline for comparison. It is estimated that implementation of the results of this study will save approximately $29 million (capital plus maintenance over 20 years) in Sanitation Districts' facilities.

Organics

The factor of greatest concern in the indirect potable reuse of wastewater is stable organic material and its potential chronic health effects on humans. Organic material in reclaimed water may also affect industrial reuses requiring low-color content process water. Pilot plant studies on the applicability of granular activated carbon for removal of dissolved and suspended organic matter from secondary effluent have been conducted at the Districts' Pomona research facility during the last ten years (9). Various system configurations containing up to four columns of granular activated carbon and providing total contact times of 20 to 40 minutes have been tested. The thrust of the experiments was to determine the degree of color and chemical oxygen demand removal that could be achieved by treating secondary effluent with granular activated carbon.

Based on the experiments, it was concluded that the first stage carbon column could provide both filtration and adsorption as long as the column was backwashed daily. This was an important finding in that it indicated that pretreatment of the secondary effluent by inert media filtration was unnecessary. Regeneration of the carbon on site was also demonstrated to be a cost-effective process.

Average water quality produced by a four-stage carbon system is summarized in Table IX. The four-stage system provided a total contact time of forty minutes. With respect to organics removal, the system produced an effluent total COD of 10 mg/l and color of 3.0 units. These data were generated through four years of research on the carbon system, which included four regeneration cycles.

The removal efficiency of carbon systems is directly related to cost. When carbon is regenerated more frequently, organic removal efficiency is higher, but the carbon replacement and regeneration operational expenses are also higher. For example, in Table X, the efficiency and cost of carbon systems with 10-minute and 30-minute contact times are compared based on achieving 10 color units in the effluent or 50 percent COD removal. For equivalent color removal down to 10 units, the 10-minute contact-time system can be regenerated every 45 days at a carbon dosage of 700 lbs. per million gallons treated at a cost of 12.1¢ per thousand gallons. Whereas, for COD removal equivalent to the 30-minute system, regeneration must be accomplished every 15 days at a cost of 20.5¢ per thousand gallons. Selection of the optimum carbon contact system is dependent on the objectives to be achieved and a careful analysis of capital and operation and maintenance (O & M) costs.

Volatile Chlorinated Organics

Experiments were conducted to determine the removal of volatile organic compounds by the treatment systems depicted in Figure 3. The objective of these experiments was to characterize secondary and tertiary effluents in terms of the six volatile chlorinated organic compounds monitored in an 80-city survey of domestic water supplies conducted by the USEPA (10). In the experiments conducted at Pomona, samples were collected at the points indicated in Figure 3. Samples were collected in 30 ml septum vials according to EPA techniques (11). Samples were analyzed according to the EPA procedures of Bellar & Lichtenberg (12) using a Hall electrolytic conductivity detector.

TABLE VIII

ESTIMATED COSTS OF TREATMENT ALTERNATIVES

SYSTEM	DISINFECTANT RESIDUAL OR DOSAGE,[1] mg/l	CAPITAL ¢/1000 gal	O&M ¢/1000 gal	TOTAL ¢/1000 gal
B-Cl_2	10	7.1	6.6	13.7
C-Cl_2	5	9.5	6.5	16.0
D-Cl_2	4	11.7	8.2	19.9
A-Cl_2	5	9.9	11.6	21.5
B-O_3	50	10.0	11.8	21.8
A-O_3	10	10.4	13.1	23.5

Notes:

1. In systems employing chlorination, the two hour chlorine residual is reported.

   In systems employing ozonation, the dosage is reported.

2. Plant size = 10 MGD

3. Dechlorination is included in cost estimate

4. ENR = 2500. Capital is amortized over 20 years at 7% interest.

TABLE IX

PERFORMANCE OF FOUR STAGE ACTIVATED CARBON SYSTEM

Parameter	Secondary Effluent	Carbon Effluent	Removal %
Suspended solids, mg/1	9	0.6	93
Turbidity, FTU	5	1.2	75
Color	28	3	89
Total COD, mg/1	43	10	77

Total carbon contact time = 40 minutes

TABLE X
CARBON ADSORPTION

A. **WATER QUALITY PARAMETERS:**

CONTACT TIME	Dissolved COD (mg/l)			Color (Units)			Turbidity (FTU)		
Minutes (Dosage)	Inf.	Eff.	% Removal	Inf.	Eff.	% Removal	Inf.	Eff.	%Removal
10 (700 lbs/mg)	30	22	27	30	10	67	5	1±	76±
10 (2100 lbs/mg)	30	15	50	30	8	73	5	1±	76±
30 (400 lbs/mg)	30	15	50	30	10	67	5	1±	76±

B. **PROCESS COSTS:**

CONTACT TIME	Capital Cost		O & M Cost	Total Cost
Minutes (Dosage)	$\$ \times 10^6$	¢/1000 gals.	¢/1000 gals.	¢/1000 gals.
10 (700 lbs/mg)	2.5	6.2	5.9	12.1
10 (2100 lbs/mg)	3.5	8.6	11.9	20.5
30 (400 lbs/mg)	3.8	9.7	5.4	15.1

C. **ASSUMPTIONS:**

1) Hydraulic Loading = 4 gpm/ft^2.
2) Carbon regeneration loss = 7%.
3) Carbon regeneration fuel rate = 6000 BTU/lb of carbon.
4) Amortization of capital at 7% interest over 20 years.

The six compounds that were analyzed were:

1. Chloroform
2. Carbon tetrachloride
3. Bromodichloromethane
4. Dibromochloromethane
5. Bromoform
6. Dichloroethane

Of these six compounds, chloroform and carbon tetrachloride were the most frequently isolated. The brominated species were so infrequently isolated in secondary and tertiary effluents that removal by the treatment systems of these compounds was not ascertained. In analyzing the results of the volatile organics research, data on chloroform were treated in greater detail than the other organics because it was the most frequently isolated and found in highest concentrations. Results of chloroform analyses for the treatment systems depicted in Figure 3 are summarized in Table XI. The most dramatic comparison to be made, based on the data in Table XI is the change in chloroform concentration during chlorination. Systems A, B, and C utilized combined chlorine residuals. The chloroform concentration increase through chlorination in these three systems ranged from five to six fold. Combined residuals employed in these experiments ranged from 5 to 8 mg/l. The chlorine contact time was two hours. As indicated in Table XI, the increase in chloroform concentration through free residual chlorination in System D was markedly higher than had been observed in combined residual chlorination. The average increase in chloroform concentration through free residual chlorination was approximately 330 fold. The filtered effluent in System D contained approximately 0.6 μg/l of chloroform. The average concentration after free residual chlorination, using free residuals ranging from 2 to 5 mg/l and a two-hour contact time, was increased to 200 μg/l.

The effects of ozonation on chloroform concentration are also indicated in Table XI. Two of the four systems showed a slight decrease in chloroform concentration through ozonation, and the remaining two systems showed a slight increase. These data indicate that ozonation has little effect on chloroform concentration.

Systems A and C employed carbon treatment after chlorination. Chloroform removal in the ten-minute carbon contact period in System C was approximately 80 percent. In System A, where a 20-minute carbon contact period was employed, the removal was approximately 90 percent. Experiments were not conducted at Pomona to determine the optimum placement of activated carbon treatment relative to chlorination. It may be optimum to remove the precursors of chloroform and other chlorinated organic materials prior to chlorination.

Inorganic Constituent Removal

Inorganic constitutents that may be of concern in water reuse systems include nitrate, phosphorus, metals, and total dissolved solids. Nitrate would normally be a concern in groundwater recharge for potable reuse. However, because of the tremendous dilution that occurs in the recharge system, nitrate has not been a concern here. Some research on nitrogen removal by tertiary treatment systems has been conducted by the Sanitation Districts. The research centered on biological methods involving nitrification followed by denitrification. Three modes of nitrification have been investigated by the Districts. A 1.5-mgd two-stage activated sludge nitrification system was operated for a period of three years. The first stage provided removal of organic material and the second stage was operated to accomplish nitrification. In a separate study, a 30-gpm pilot scale, fixed growth nitrification column was operated for one year. In this system, secondary effluent was fed to gravel columns and air or pure oxygen was injected to create aerobic conditions. A third method of nitrification has been investigated in full-scale water reclamation plants. Nitrification in conventional single-stage activated sludge systems was studied at the Whittier Narrows (15 mgd) and San Jose Creek (30 mgd) Water Reclamation Plants. All three nitrification methods produced well-nitrified effluents with ammonia concentrations of less than 1 mg/l (as N). The single-stage activated sludge system proved to be the most cost-effective method for nitrification in the temperate climate of Southern California. It involved reduction of plant flow by approximately 25 percent while maintaining the same air input rate to aeration basins.

Biological denitrification on both

TABLE XI

SUMMARY OF CHLOROFORM MONITORING

System	Chloroform Concentrations, µg/l Secondary Effluent	 Predisinfection Effluent (A)	 Disinfected Effluent (B)	Change in Chloroform During Disinfection (B/A)
A-Cl_2	1.2	0.8	4.8	6.0
A-O_3	1.2	0.8	0.5	0.6
B-Cl_2	1.2	1.5	9.1	6.1
B-O_3	1.2	1.5	1.3	0.9
C-Cl_2	0.4	0.9	4.6	5.1
C-O_3	0.4	0.9	1.3	1.4
D-Cl_2	0.3	0.6	198	330
D-O_3	0.3	0.6	0.7	1.2

inert media and activated carbon columns has been evaluated in pilot studies conducted by the Districts (13). Methanol was used as a source of readily available organic materials for the denitrifying bacteria. Denitrification efficiencies of 85 to 90 percent were achieved at column detention time of 10 minutes. Both carbon and inert media performed with equal efficiency.

For recreational reuses that involve impoundment of the reclaimed water in lakes removal of phosphorus is necessary to prevent algae growth. Studies of phosphate removal have been conducted both at the Pomona research facility (14) and at the Districts' Lancaster water renovation plant (15).

Research at Pomona demonstrated that it was possible to obtain a high degree of phosphate removal from wastewater by the addition of ferric chloride or alum to the aeration basin of an activated sludge plant. Alternatively ferric chloride or alum could be used as a coagulant in a tertiary treatment system. The study indicated that organic removal efficiency of the activated sludge process would not be adversely affected by alum doses as high as 200 mg/l, or ferric chloride doses up to 130 mg/l, but that effluent suspended solids concentrations would be increased. Addition of an inert media filter to the pilot treatment train was necessary to consistently achieve effluent suspended solids concentrations of less than 5 mg/l. The treatment train of alum addition to the aeration basin followed by secondary clarification and then by inert media filtration produced an effluent total phosphate concentration in the range of 0.5 to 1.2 mg/l (as P).

Pilot scale research at Lancaster indicated that both algae and phosphate removal can be effectively achieved by using 200 to 300 mg/l alum as the primary coagulant in a coagulation-sedimentation-filtration process for renovating oxidation pond effluent.

The total dissolved solids (TDS) concentration of water is increased by 300 to 400 mg/l during domestic use, which may create the need for demineralization to meet TDS requirements for groundwater recharge, irrigation, or industrial reuse. Of the various methods of demineralization available, the Sanitation Districts have investigated reverse osmosis, electrodialysis, and ion exchange. During reverse osmosis experiments, the RO membranes were fed both secondary effluent and activated carbon treated effluent. Results indicated that TDS removal efficiencies of 90 to 95 percent and water recovery efficiencies of 75 to 80 percent could be achieved. However, the RO membranes required frequent cleaning to prevent fouling due to biological growths supported by organic matter in wastewater. The best membrane life demonstrated in these experiments was one year.

In experiments with electrodialysis, the feedwater consisted of activated carbon treated secondary effluent. TDS reductions of less than 35 percent were achieved in a single pass electrodialysis system necessitating a series of units to achieve high TDS removal. The electrodialysis process was severely hampered by biological fouling and required weekly cleaning to maintain demineralization efficiency. Because of this, electrodialysis was the most expensive of the three systems investigated.

Ion exchange systems evaluated by the Districts included a two-stage countercurrent fixed bed and a single-stage counter-current moving bed system. Each system was fed carbon treated secondary effluent. Both systems removed approximately 90 to 95 percent of the influent TDS. Ion exchange is the least expensive of the three demineralization methods studied.

By definition, demineralization processes remove metal ions effectively. Other tertiary treatment systems researched by the Sanitation Districts removed metals comparatively inefficiently by removing particulates to which metals had been adsorbed. However, the secondary effluents presently produced by the Districts' plants routinely meet drinking water standards for metals, obviating concern over removal by tertiary treatment systems.

APPLICATION OF WATER RECLAMATION TECHNOLOGY

Direct Filtration

In order to meet the unrestricted recreational impoundment requirements of the California Administrative Code (Title 22), filtration systems are being constructed at four secondary treatment plants that discharge to the San Gabriel and Rio Hondo Rivers. The designs employ dual- or multimedia filtration and will incorporate both chlorination and dechlorination. Dechlorination will be achieved with sulfur dioxide. A unique feature of the filtration system design is that the filters are being constructed with sufficient depth to allow substitution of activated carbon for inert media should the need develop to remove additional trace organics at those facilities whose effluent is used for groundwater recharge.

At another of the Districts' secondary treatment plants located inland in a desert area (Valencia, California), a direct filtration system has been operated for approximately one year. The system consists of low-dose alum coagulation followed by multi-media filtration and disinfection with combined residual chlorination. Dechlorination is also included to protect aquatic organisms. Design data for the direct filtration and disinfection units are summarized in Table XII.

Secondary effluent quality and final disinfected effluent quality data are briefly summarized in Table XIII. These data were accumulated over a 12-month period during which daily samples were collected for each of the parameters reported. As indicated, the direct filtration system results in a highly clarified tertiary effluent with turbidity of approximately 1.2 FTU. This well-clarified effuent in combination with the chlorine contact time of eight hours has resulted in consistent achievement of effluent total coliform median concentrations of less than 2 MPN/100 ml, which is the limit specified in Title 22 of the California Administrative Code. Coliform inactivation has been so efficient in the filtered effluent at this plant that, in daily samples collected since March 1976, coliform concentrations greater than 2 MPN/100 ml have been detected in less than 25 samples.

The direct filtration system has been in operation for approximately one year, which has provided an opportunity for determining the operational and maintenance costs of this system. The estimated operational and maintenance cost of the direct filtration system, including chlorination and dechlorination, is 5.7¢/1000 gallons. The estimated capital cost of the system is 6.9¢/1000 gallons based on the $1.0 million initial cost (1973 dollars). Therefore, the total cost of direct filtration and chlorination/dechlorination is estimated to be 11.5¢/1000 gallons.

Carbon Filtration - Pomona

A 10-mgd single-stage granular activated carbon filtration system has been constructed at the Pomona Water Reclamation Plant. Although the system was designed to provide virus removal, it will also produce effluent that will be usable in a paper processing plant, which requires low color effluent. Virus removal was required to meet Health Department requirements prior to discharge to an open river channel used for recreational purposes. The carbon contactors are oeprated in gravity down-flow mode. The design hydraulic loading rate is 3.5 gpm per sq. ft., and the depth of the carbon in the contactors is 6 ft., which will provide an empty bed contact time of ten minutes. The particle size of the granular carbon is 8 x 30 mesh. The carbon will be regenerated using a 54" diameter six hearth furnace. The design chlorine contact time is two hours, and the chlorinated effluent will be dechlorinated using sulfur dioxide. A summary of operation parameters for the system is contained in Table XIV.

The plant has been in operation since January 1977. Effluent quality data generated in the first month of operation are summarized in Table XV. As indicated, low-effluent suspended solids and turbidity levels are achieved through the one-stage carbon filtration system. When the operation is stable, it is expected that the effluent chemical oxygen demand will be approximately 22 mg/l, and the effluent color will be ten units. The data shown in Table XV were collected prior to the time when regeneration would normally have occurred, and hence, the COD and color levels are slightly lower than would be expected at steady state. The effluent produced by the carbon system is well prepared for disinfection as indicated by the total coliform count of less than 2 MPN/100 ml using a 4 mg/l chlorine residual.

Insufficient operating data prevent determining actual operation and maintenance

TABLE XII

SUMMARY OF OPERATION PARAMETERS
DIRECT FILTRATION SYSTEM - DISTRICT 32

Inert Media Filters

Plant Flow	2.5 MGD (design = 4.5 MGD)
Number	3
Diameter	10 ft.
Hydraulic Loading Rate	3.5 gpm/ft^2
Backwash Rate	20 gpm/ft^2
Backwash Frequency	24 hours
Alum feed	5 mg/l
Polymer (WT-3000)	0.07 mg/l
Media (Multi-media)	Anthracite, silica sand, garnet

Chlorination

Contact Time	8 hours (design = 4 hrs)
Average Residual	2 mg/l (combined)
Dechlorination (sulfur dioxide)	18 minutes

TABLE XIII

AVERAGE EFFLUENT QUALITY
DIRECT FILTRATION SYSTEM - DISTRICT 32

Parameter	Average Secondary Effluent Concentration	Average Tertiary Effluent Concentration
Suspended Solids, mg/l	12	1.6
Turbidity, FTU	4.3	1.2
Chemical Oxygen Demand, mg/l	[illegible]3	23
Total Nitrogen, mg/l-N	N.D.	12
Total Coliforms, MPN/100 ml	N.D.	<2 (median)

TABLE XIV

SUMMARY OF OPERATION PARAMETERS
ACTIVATED CARBON TREATMENT - POMONA

ACTIVATED CARBON FILTERS

Plant flow	10 MGD
Number	4
Hydraulic Loading Rate	3.5 gpm/ft^2
Carbon depth	6 ft.
Empty bed contact time	10 min.
Backwash rate	15 gpm/ft^2
Backwash frequency	24 hours
Granular carbon size	8x30 mesh

CHLORINATION

Contact time	2 hours
Average Residual	4 mg/l (combined)
Dechlorination (sulfur dioxide)	(injected ahead of effluent weir)

TABLE XV

AVERAGE EFFLUENT QUALITY
CARBON ADSORPTION SYSTEM - POMONA WRP

(Data generated in first month of operation)

Parameter	Average Secondary Effluent Concentration	Average Tertiary Effluent Concentration
Suspended Solids, mg/l	7	1.1
Turbidity, FTU	N.D.	1.3
Chemical Oxygen Demand, mg/l	45	20
Color	26	4
Total Coliforms, MPN/100 ml	N.D.	<2

costs of the carbon and chlorination/dechlorination system. Based on pilot scale work, it is estimated that the operation and maintenance cost will be 6.5¢/1000 gallons. The capital cost of the carbon and chlorination/dechlorination systems was $3.3 million (1974 dollars). Amortizing this capital expense over 20 years at 7 percent interest, the capital cost of one-stage carbon treatment and chlorination/dechlorination is approximately 8.5¢/1000 gallons. Therefore, the total cost of this treatment system is approximately 15¢/1000 gallons.

FUTURE RESEARCH

The consultant panel to the State of California on health aspects of water reclamation for groundwater recharge (16) made recommendations related to research on the health effects of groundwater recharge with reclaimed water. These recommendations included:

(i) Waters produced by an established, well-engineered, and operated groundwater recharge system should be quantitatively analyzed to provide a data base for estimating health risks.

(ii) These quantitive analyses should be coupled with epidemiological studies of populations consuming the water source.

In an attempt to implement these recommendations, the Los Angeles County Sanitation Districts have proposed a study of the health effects of groundwater recharge. As described previously, intentional water reuse by surface spreading and percolation of diluted secondary effluent has been practiced in the Montebello Forebay since 1962. Records of quantity and quality of reclaimed water spread since that time exist. Because the Montebello Forebay project is well controlled and the oldest and largest intentional reuse system in the state, it provides an ideal situation for health effects research.

The research, which is to begin in April 1977, contains tasks dealing with hydrogeologic studies, detailed water quality characterizations, epidemiology, and toxicology. The purpose of this study is to determine whether chronic health effects due to constitutents in wastewater can be detected. The epidemiological approach to be taken in this study will not eliminate potential detection of health effects due to inorganics of viruses; however, these constituents are not expected to be of health concern in this water reuse system. The hydrogeological studies will be conducted to determine the impact of reclaimed water spreading on each water supply well in the area. The detailed water quality characterizations will include monitoring for trace organic compounds using GC/MS techniques, monitoring for low concentrations of virus, and routine water quality analyses on wellwater, reclaimed water, and imported water supplies. The epidemiological studies are in a formative stage at this time, but will potentially include a multi-faceted approach including review of existing statistics, and possibly, surveys. The toxicological research will include use of the Ames *Salmonella* mutagen test (17), and possibly, laboratory animals. It is expected that the study will span three to five years and that the project cost will be greater than one million dollars. The expected result of the study is a determination of whether or not the existing water reuse system can continue and/or be expanded.

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